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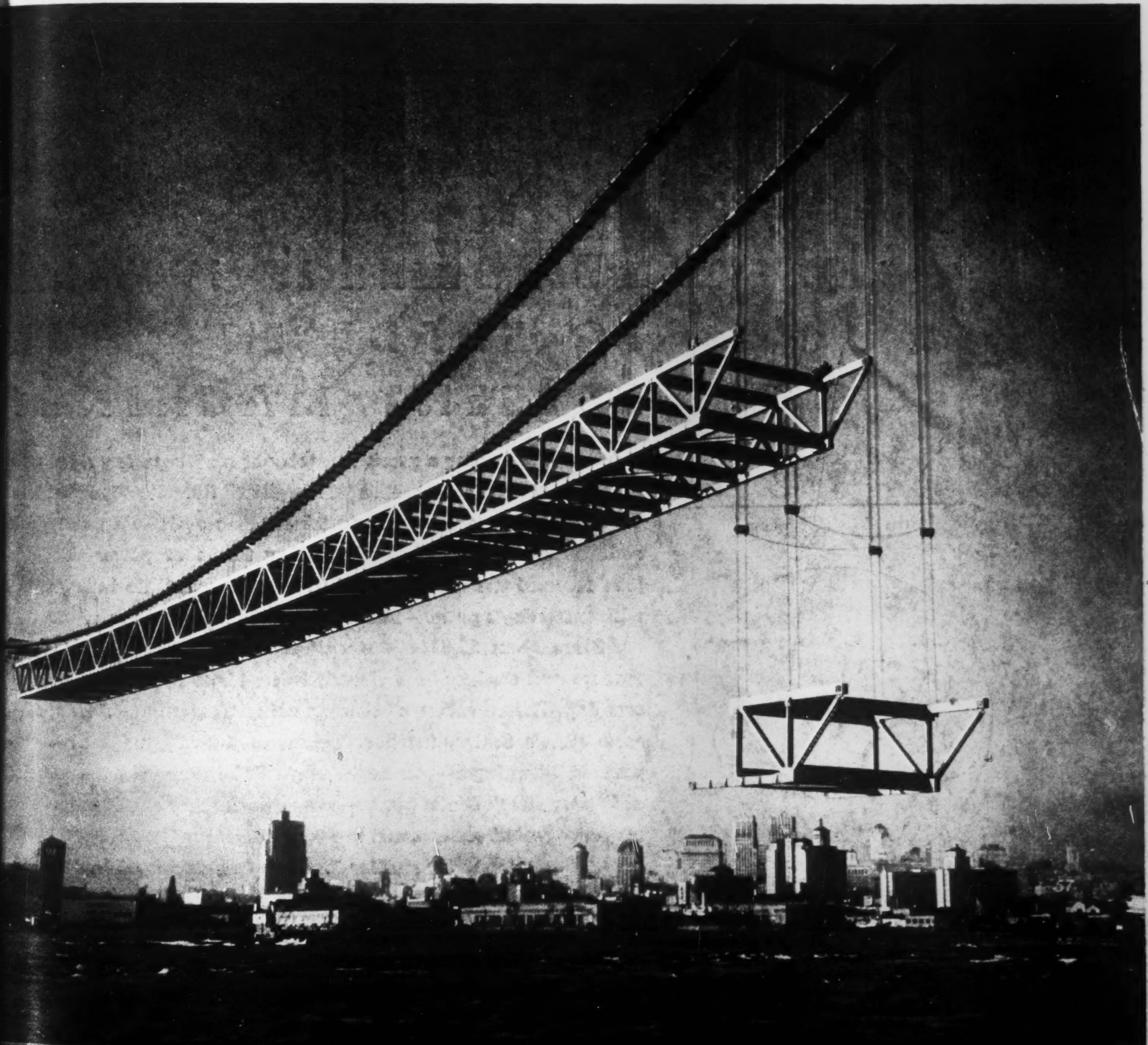

A truss unit for one of the suspended spans of the San Francisco Bay Bridge being hoisted to place. Barges bring the units out under the lifting beams that advance along the cables.

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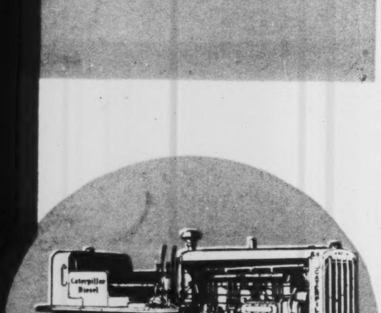

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A MOMENT WITH THE PUBLISHERS

America meets *the Map*

WHEN WE cast up the manifold values that have been created by the motor age, we are quite likely to overlook one that may be minor in its economic significance but that surely is as universally felt as any other. Probably that very universal quality leads us to take it for granted and so overlook it.

Not so long ago the average American would shy from a map as quickly as he would from an abstruse mathematical chart. Maps reminded him of geography and that was something he had to study in school and, therefore, something he was glad to have behind him. And as for a woman—well, who would expect her to make head or tail of so technical a matter as a map?

The only maps that really had any general use at all were those weird layouts in the railroad timetables, no two of them alike for the same area because each was distorted to show its own road straight as the crow flies, let rivers, mountains, cities and state lines fall where they might. Even those "maps" rarely were consulted by the average traveler.

But today all that is changed. Pull up alongside any car halted momentarily at a crossroads anywhere in the U. S. A., and you will find father, mother, sister and even little Willie deeply engrossed in a map and learnedly interpreting its indications with every evidence of intimate familiarity and cordial appreciation. No American family is complete without its road maps.

NOT LONG ago a sweet young thing who drives thousands of miles every summer to beguile her holidays confided to this writer that it was too bad that "maps looked so flat and couldn't show the different mountains and everything like that." When it was explained to her that maps are made to do exactly that, she was as tickled as a small boy with a new cowboy suit. Indeed, the motor and the highway have made the American people map-minded.

With all this has come marked improvement in the technique of map making and distribution. No American family need be mapless so far as its own country is concerned. The oil companies, of course, have been leaders, but the states highway departments, the motor clubs and other agencies have done a fine job. Today, wherever you may be, a good road map is just around the corner.

Further progress is in the making. Having gone so far as to indicate the most negotiable routes from A to B, our popular maps are beginning to show something of the country between. Points of natural and historic interest are indicated by notes and in some cases topography is suggested in fairly acceptable fashion.

DOUBTLESS more and more attention will be devoted to the major drainage features and to other aspects of topography. Many motorists are beginning to show a lively interest in such features. And the railroads—there is a chance for improvement. Few road maps show the railroads; yet these frequently are valuable landmarks to the driver. Many would be glad to have information also as to whether the crossings are at grade, over or under. All this is written in no spirit of captious criticism for a job so admirably done in so short a time, but rather to indicate some of the more obvious lines open to further progress.

Underlying all that the commercial map-makers can do are the basic surveys and mapping that must be conducted under the auspices of state and national governments. This work, of course, serves many functions that are more substantial and fundamental than that of making road maps. But it would seem that with the rank and file of the people so map-conscious as they are today, we should be able to enlist more popular support for the map-making activities of government.

No longer are these of interest only to a few scientists and engineers; they have become an everyday interest and service to the everyday American family.

Willard Chevalier

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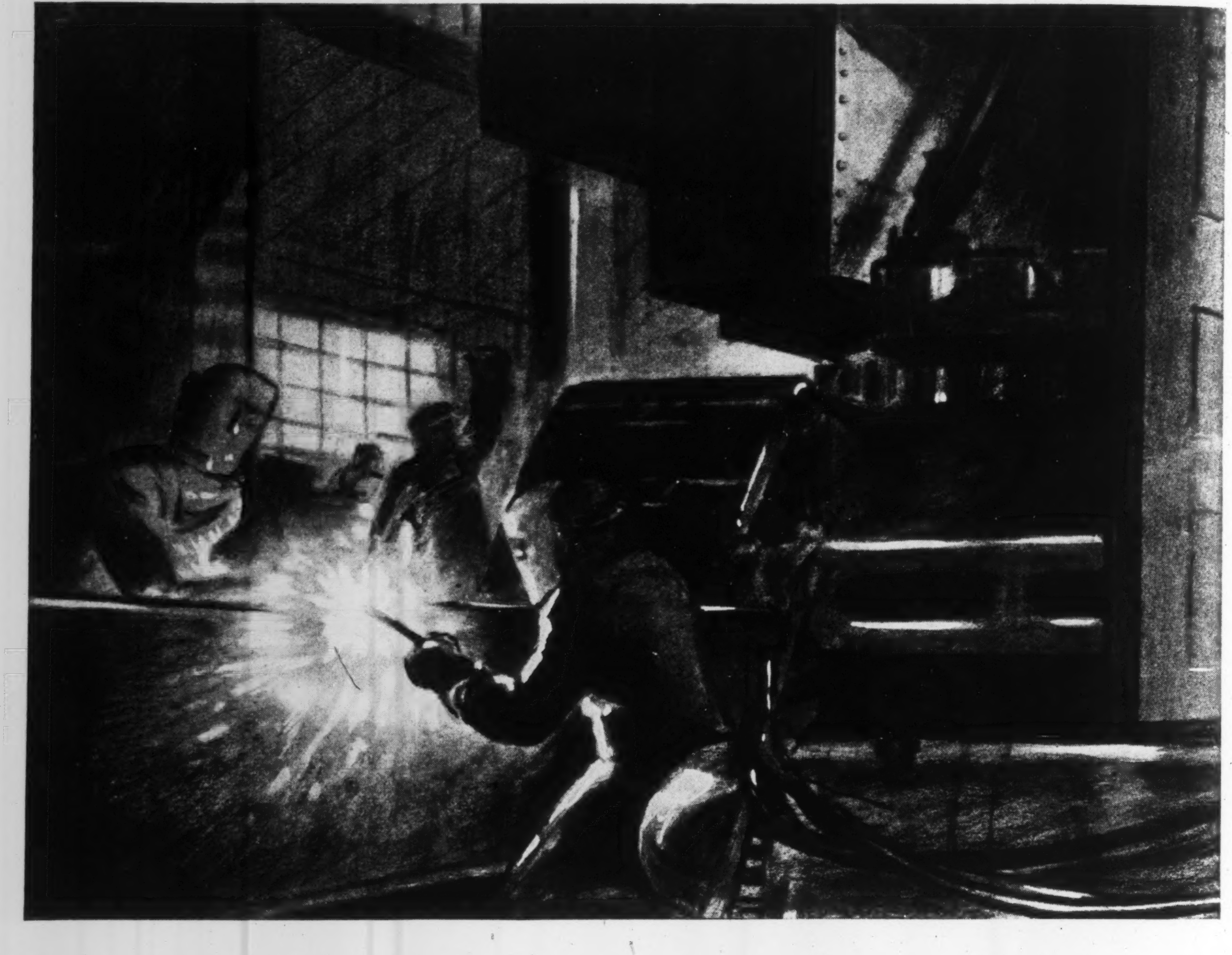

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■

Speeding Superhighways—SUPERHIGHWAYS IN GERMANY are built at top speed as part of a 5,000-mile system. The system and some of the interesting engineering involved will be discussed in two articles to be published next week.

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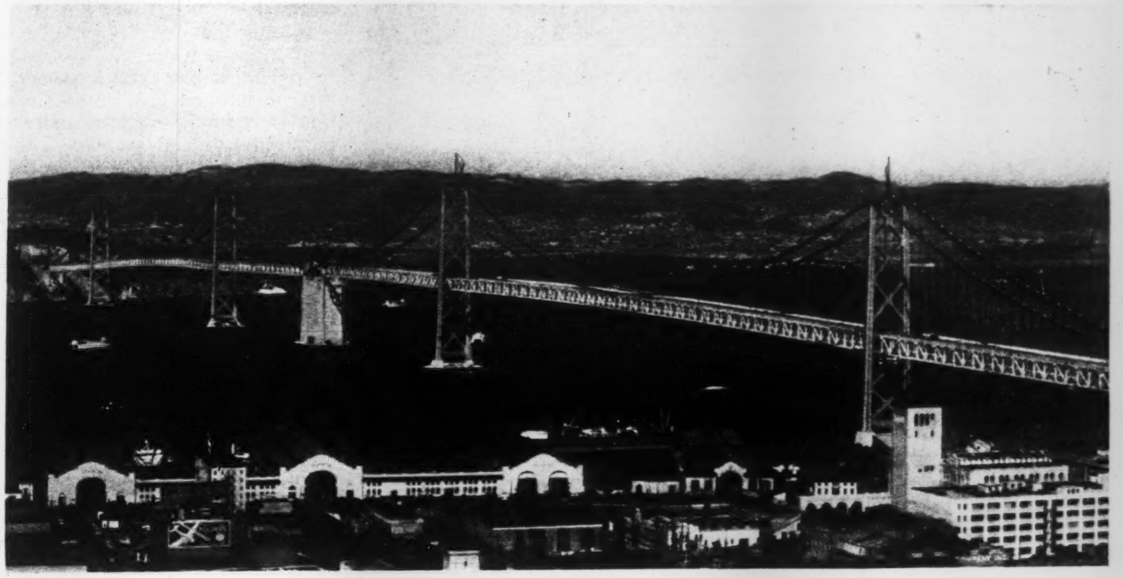


FIG. 1—APPROACHING COMPLETION, the Bay Bridge now is a dominant feature of San Francisco's water front

Bay Bridge Suspended Structure

On San Francisco-Oakland Bay Bridge long side spans increase flexibility. Trains make loading unsymmetrical—Assembly line deck units lifted bodily to place

By C. H. Purcell, Chas. E. Andrew and Glenn B. Woodruff

Respectively, Chief Engineer, Bridge Engineer and Engineer of Design, San Francisco-Oakland Bay Bridge

THE WEST BAY crossing of the San Francisco - Oakland Bay Bridge consists of two suspension bridges, placed end to end with a common anchorage between them near mid-channel. The two bridges over the 8,300-ft. waterway are not identical because foundation conditions made it necessary to locate the San Francisco anchorage 863 ft. beyond the west end of the west suspension bridge. The unloaded back-stay between the anchorage and the end of the side span makes the west suspension bridge somewhat more flexible than the other. This causes slightly higher stresses in the trusses of the west bridge, but not enough to warrant any difference in sections.

The principal dimensions of the Bay Bridge suspension spans and for other major suspension bridges are given in the table on p. 257.

The generally accepted measure of size of such bridges is the length of center span and in this respect the Bay Bridge stands third. The length of its side spans, however, exceeds that of any other bridge. In some ways this is not an advantage, since it results in greater flexibility. The federal government, however, required minimum channel widths of 1,000 ft. and a lesser side span length could not be used.

Another factor that further increases the flexibility of the structure, but which at the same time represents economy of design, is the relatively high ratio of live to dead load. Insofar as the bridge as a whole is concerned, this comparative flexibility is not a matter of consequence. It has been troublesome only in the provision of adequate expansion details, particularly for the interurban railway lines.

In the design of the roadway slabs, stringers and floor beams, truck and train loads were used with an impact factor of 33 per cent. The upper deck was designed for 10-ton trucks and the lower deck for 30-ton trucks and 70-ton interurban cars. A factor of 85 per cent was used when all lanes were loaded.

The cables, towers and anchorages were designed for the following uniform live load in pounds per linear foot:

		North Truss	South Truss	Totals
Upper deck		600	600	1,200
Lower deck	Highway	1,400	600	2,000
	Interurban	1,000	2,800	3,800
Totals		3,000	4,000	7,000

Unequal distribution of loads to the two trusses results from the location of the tracks of the interurban railway on the side of the lower deck adjacent to the south truss.

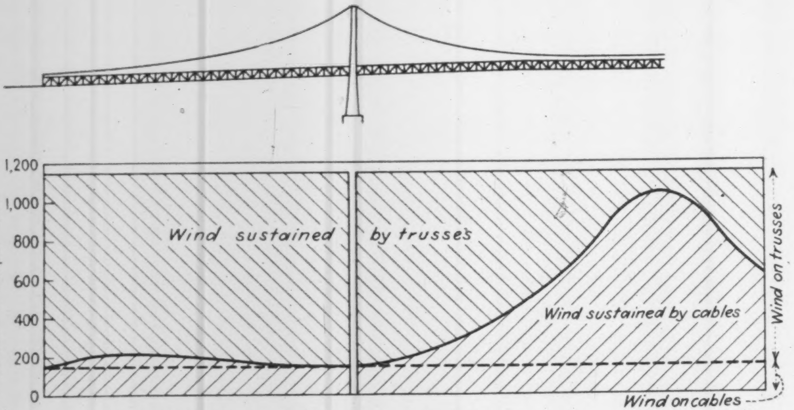


FIG. 2—LATERAL LOAD DISTRIBUTION between truss and cable on San Francisco-Oakland Bridge.

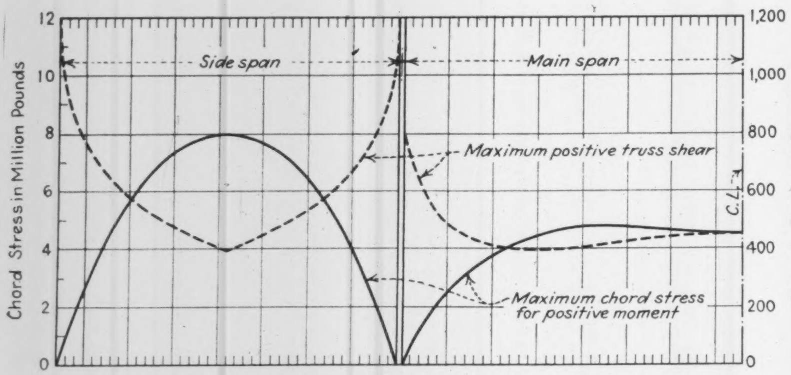
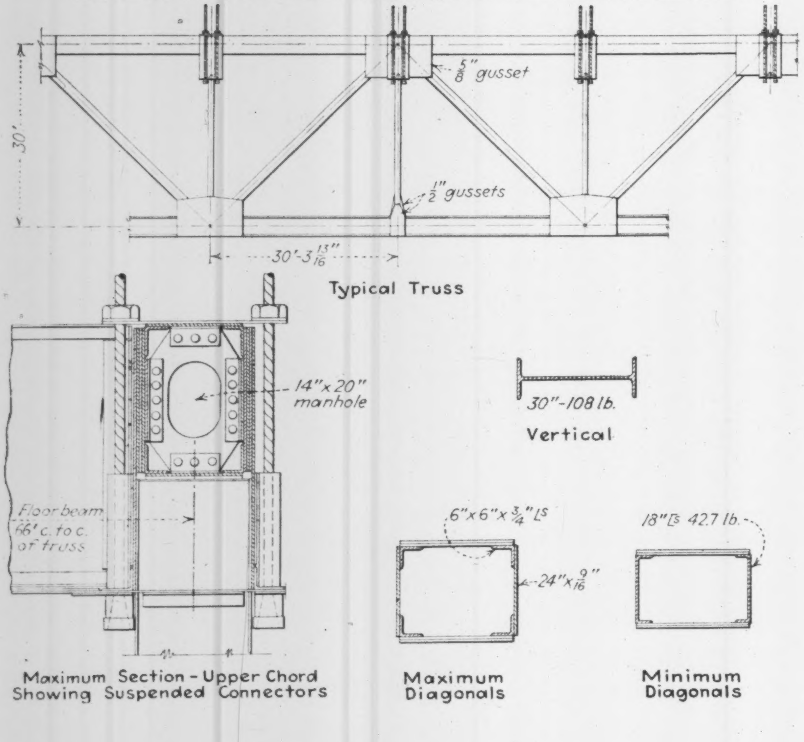


FIG. 3—CHORD STRESSES AND TRUSS SHEARS in the suspension spans.

FIG. 4—FLOOR SYSTEM details showing typical truss and suspender connectors.



Since the live load is greater and the dead load less on the south truss, it will be subject to the greater stresses. In the design the south truss only was analyzed and both trusses were made identical.

In a suspension bridge, both maximum truss stresses and distortions are caused by loading certain spans or parts thereof, with no load on adjoining sections. There is no probability of the occurrence of such a limited loading, so in computing deflections an "unbalance" of 3,000 lb. per linear ft. was assumed; i.e., 4,000 lb. on those sections causing maximium values and 1,000 lb. on the remaining portions, or 3,000 lb. only on the former. For maximum truss stresses the loading used was an unbalance of 1,500 lb. over the entire span (main or side) plus an additional 1,500 lb. over a limited portion of the span, combined with temperature and a 20-lb. wind on the bridge and moving load.

The local climatic conditions permitted the low assumption of ± 30 deg. F. from a normal of 62 deg. F.

Design analysis

As a preliminary in the design of the stiffening trusses, stresses were calculated by the elastic theory. Coefficients applied to the results thus obtained gave close approximations to the final stresses. The final analysis was made with the more accurate deflection theory. The usual application of the latter theory neglects the effects of suspender elongation, tower stiffness and the change in span lengths which results from the longitudinal deflection of the towers. These effects were analyzed and found to be of small consequence, except that the elongation of the long suspenders adjacent to the towers materially increases the truss stresses in that vicinity. Corrections for truss shears were the greatest, and amounted to a maximum of about 40 per cent.

The distortions of the structure under vertical loads were calculated by a modification of the method devised by Allston Dana in the design of the Delaware River Bridge. It is based on calculating the changes in span lengths, cable sags, and balance of horizontal forces at all towers and checking by the fact that the horizontal distances between anchorages remains unchanged. This method is applicable only for continuous loads covering one or more spans. For this particular purpose, however, where the deflections are relatively large, it is more accurate than the deflection theory.

The stresses from wind loads were calculated by the theory developed by Moisseiff and Lienhard, (*Transactions* AM.Soc.C.E., Vol. 98, p. 1080). Fig. 2 shows the distribution of the lateral load between the truss and cable.

The various theories mentioned above were all checked by an exhaustive model

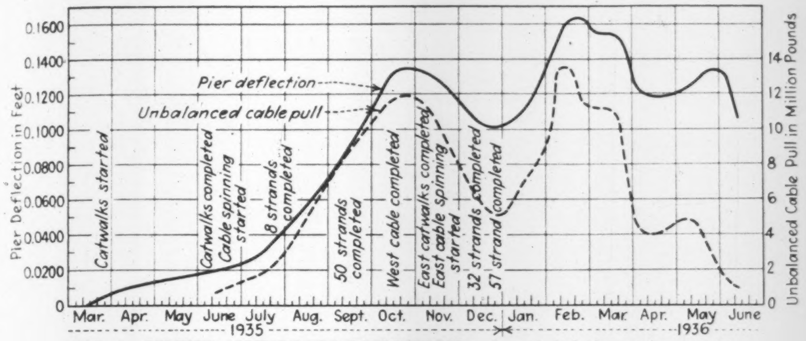


FIG. 5—CENTER ANCHORAGE DEFLECTION as it varied during various stages of erection up to the start of paving.

PRINCIPAL DIMENSIONS OF MAJOR SUSPENSION BRIDGES

	Bay Bridge	Geo. Washington	Golden Gate	Delaware	Ambassador
Span length—main span (ft.)	2,310	3,500	4,200	1,750	1,850
Span length—side span (ft.)	1,160	650	1,125.4	716.7	817.4[3]
Cable sag (ft.)	231	325	475	196.7	209.1
Depth of truss (ft.)	30	41.16[4]	25	28	22
Width of truss (ft.)	66	106	90	89	59.5
Dead load per ft. (lb.)	18,700[1]	39,000	22,100	26,000	6,200
Live load per ft. (lb.)	7,000[2]	8,000	4,000	12,000	3,300
Ratios:					
Side span/main span	1:1.99	1:5.4	1:3.7	1:2.44	1:2.26[3]
Cable sag/span length	1:10	1:9.3	1:8.85	1:8.9	1:8.9
Truss depth/span length	1:77	1:85	1:168	1:62.5	1:84
Truss width/span length	1:35	1:33	1:46.6	1:19.7	1:31.1
Live load/dead load	1:2.67	1:4.87	1:5.53	1:2.17	1:1.88

[1] 9,000 lb. to south truss and 9,700 lb. to north.
[2] 4,000 lb. to south truss and 3,000 lb. to north.
[3] Side spans not supported by cables.
[4] When built for future lower deck; only top chord erected now.

analysis made at the Univeristy of California, (*ENR,* Mar. 22, 1934, p. 367). The models were built before the final design of the structure was determined and hence, in certain dimensions, did not represent the structure as built. However, they afforded a valuable check on theory.

Fig. 3 shows the chord stresses and truss shears for one-half the bridge. In distributing the load due to wind, it was assumed that all four chords were available to resist the moment. The shear due to this load is carried by the lateral system in the plane of the lower chord only. Any participation by the roadway slabs was neglected.

Details of design

Floorbeams, lower deck stringers, truss chords, diagonals, laterals and gussets used as chord splices are silicon steel, while truss verticals and all other details are carbon steel. All members are of built-up sections except the stringers and truss verticals. These are rolled sections.

Expansion details offered a problem of detailing that required considerable thought. The flexibility of the structure will produce rather large movements between the ends of the trusses and the towers or anchorages. At these points the trusses are held in place vertically by a built-up link pinned at its top to the lower chord of the truss and at its lower end to the tower or anchorage. The trusses are secured transversely in the plane of the lower chord. The last panel of lateral K-bracing terminates in a pin connection to a steel block which slides longitudinally between vertical bronze plates secured in the cross strut of the tower. The connection is so made that the truss is free to change its transverse or vertical angularity at this point by a small amount. The movement of the block is restrained by stops so that the truss cannot move out more than 14 in. nor in more than 10 in. Only improbable loading conditions will bring these stops into action.

An inspection bridge is provided under each of the six suspended spans. These bridges are 4 ft. wide and span the full width of the trusses. They are suspended from rails bracketed on the outside of the lower chords and are propelled by hand operated cranks driving the traveling wheels through a train of gears. These inspection bridges were used as crosswalks between the footbridges during cable spinning.

Steel for the suspended structure was fabricated at the Ambridge and Gary plants of the American Bridge Co. After shop fabrication, including milling of the ends, chord sections were assembled in the yard in lengths of 8 panels, set to proper camber and, while thus assembled, holes for the chord splices were reamed. Connections for web members were reamed to metal template.

The members then were shipped to a yard in San Francisco about 3 mi. south of the bridge site. After weathering they were sandblasted, given a priming coat of red lead paint and assembled into units for erection. Each unit consisted of two full panels of both trusses with lower deck floor beams, one panel of lateral bracing and two lines of bottom stringers. When weight limitations permitted, upper deck floor beams and a panel of upper and lower deck stringers were added.

To assemble the truss units, the contractor built two parallel standard gage railroad tracks, on 66-ft. centers (corresponding to the distance between trusses). These tracks were perpendicular to the shore line and each terminated on a timber pile trestle extending a short distance into the bay. Two stiffleg derricks were set up at the shoreward end of the tracks. One derrick had a 30-ton capacity with a 70-ft. boom; the other a capacity of 65 tons with a 75-ft. boom. These derricks assembled each unit on four 4-wheeled railroad trucks, placed so that each truck carried one corner of the unit. The unit was then moved down the assembly line where all joints that could be completed at this stage were riveted before it finally was carried out on the trestles. When ready for delivery, a barge was floated between the trestles and under the unit. By pumping the water from the hull the draft was decreased and finally the truss unit was lifted free of the tracks and supported on blocking secured to the barge. It was then towed to the bridge site for erection.

The above setup made it possible to assemble and load a maximum of three units, with an average weight of 175 tons each, in a 12-hr. day.

Erection

The general scheme of erecting the suspended steel proposed by the contractor was notably different from the usual practice on other large suspensoin bridges. In lieu of erecting by floor travelers and placing one piece at a time, two panel units, asesmbled as above described, were lifted into place from a barge by lightweight travelers or lifting beams supported directly from the main cables.

The lifting beams were built-up box sections about 2 ft. 6 in. square, inside of which were set the necessary sheaves for rigging the falls. The lead lines extended horizontally from the struts to the towers and thence vertically to the base where the hoisting engines were set. When a lift was made, these beams were suspended from the cables by 8 parts of $1\frac{1}{8}$-in. wire rope. Curved steel plates, lined with heavy belting, protected the main cables from damage by these ropes. To move the beams along the cables, the supporting ropes were removed and the weight of the beams and falls carried on wooden spools cut to fit and roll along the cable. During these moves it was necessary to shorten the beams so that they would clear the suspenders. Therefore the ends of the beams were made retractable by fitting a short length of box section inside the larger and main portion of the beam. A screw controlled this movement.

Two beams were required at each lifting point, and the falls were secured to the truss sections at four points. The total weight of the two beams, including falls, was only 38 tons. Four engines were mounted at the base of each tower and rigged to lift in the adjacent side and half main span. All four engines were required to lift one unit.

Since the hoisting engines were not synchronized, there was a tendency for the sections to get out of level while they were being lifted. This tilting was measured by a pair of pendulums set at right angles to each other. Tilting caused these pendulums to sweep across electric contacts and light bulbs on a control box in front of a signalman.

Shoreline and steamship piers made it impossible to spot the barge under portions of the San Francisco and Yerba Buena side spans. In these sections, units of two panels of one truss were lifted with single sets of falls lashed to the cables. One set of these falls was directly over the point where the section was to be finally placed. A second set was placed so that it could lift the unit from the barge. In the air and above all obstructions the unit was "telegraphed" from one set of falls to the other and lifted into place.

For several reasons it was desirable to erect as many units as possible at one point before shifting to another. The following controls were therefore established:

1. The weight of the unit was kept sufficiently low so that the concentration at the suspender points would not cause an angular change in the cable in excess of that in the completed bridge. Since this angle is a function of the relation between the concentrated load and the tension of the cable, it was necessary to restrict the weights of the first units raised to not more than 150 tons.

2. The bending of the towers was kept within limits which would not stress the anchor bolts to exceed 5,000 lb. per sq.in. nor produce a combination of bending and direct stress in the extreme fiber of the tower leg of more than 28,000 lb. per sq.in. The permissible deflection varied among the towers and with the vertical load on them. The maximum deflection permitted in towers 3 and 5 (the tall towers) was 5 ft. and in towers 2 and 6 was 4 ft. 2 in.

3. The unbalanced pull on the center anchorage was limited to 6,200,000 lb. per cable. This was the maximum that could be produced by the completed cables on one side of the anchorage and has been arbitrarily maintained throughout construction. While it is only 60 per cent of the unbalance for which the center anchorage was designed, this condition extends over comparatively long periods of time, and the question of plastic flow in the concrete entered. Fig. 5 shows the deflection at the top of the center anchorage during the various erection stages to the start of paving. While for the total height of structure of 450 ft. the deflections are comparatively small and are as close to theory as may be calculated, a small amount of permanent set may be noted.

The contractor erected a total of 146 truss units between December 18, 1935, and April 30, 1936; an average of 7.3 units per week. The heaviest unit erceted weighed 203.5 tons. The erection method proved very satisfactory and has several noteworthy advantages:

1. It permitted an unusual proportion of the assembly and riveting to be done in the yard instead of in the field.

2. The usual heavy floor travelers were avoided.

3. Fewer lifts were required, thus giving less interference to navigation.

4. A minimum of time was consumed.

Model aids erection

Economy in erection made it desirable that as much of the floor steel as possible be lifted with the units. The contractor's assembly line in the yard made it necessary that the order of erection for at least five units be known in advance. Furthermore, it was desirable to complete the work and release all of the lifting beams and engines around one of the towers in the west suspension

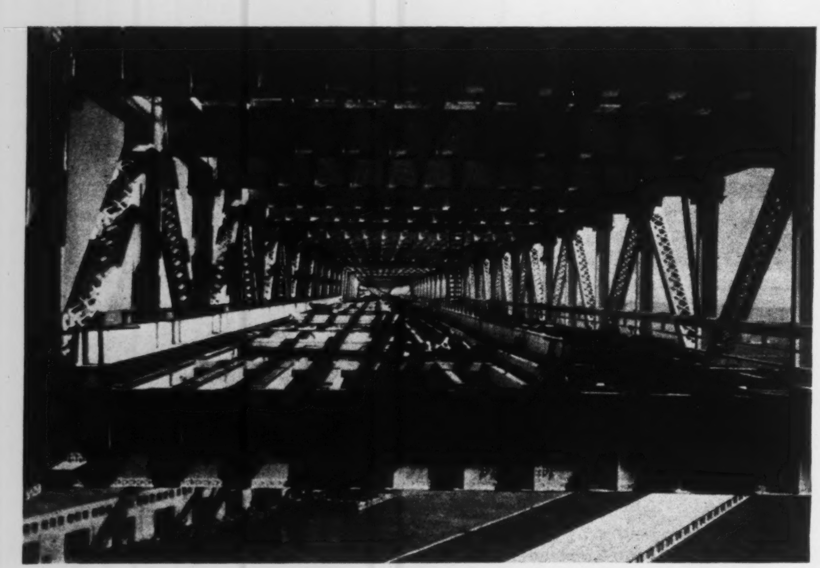


FIG. 6—LOWER DECK VIEW showing space for trucking roadway at left and for interurban tracks at right.

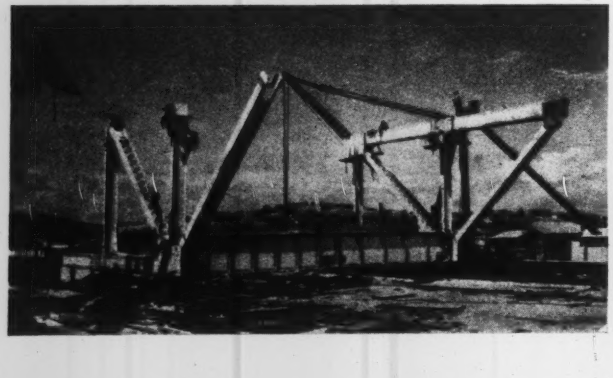


FIG. 7—TRUSS UNIT ASSEMBLY at the upper end of the assembly line.

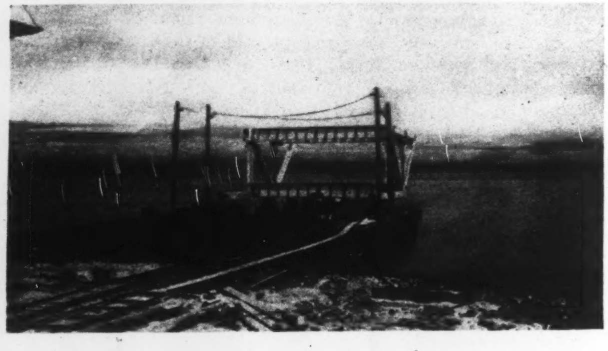


FIG. 8—BARGE LIFTING TRUSS UNIT at lower end of assembly line.

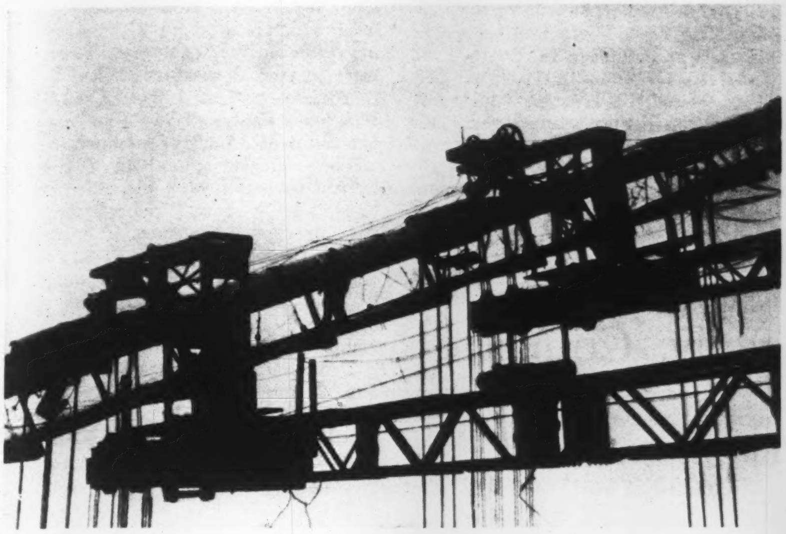


FIG. 9—LIFTING BEAMS move along the main cables on grooved wooden spools but are suspended by cable slings while lifting trusses.

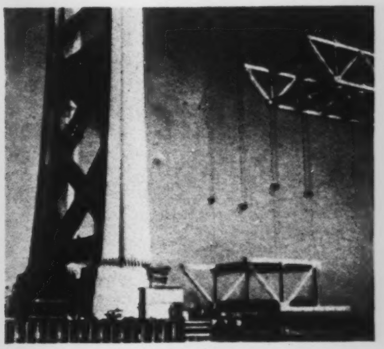


FIG. 10—UNIT BEING HOISTED from barge by tackle from lifting beams on main bridge cable.

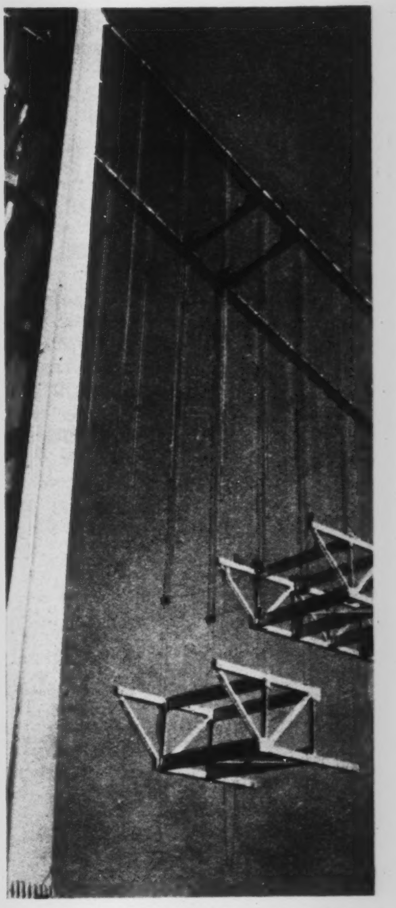


FIG. 11—A TRUSS UNIT approaching its destination where men on bridge are ready to make it fast.

bridge as much in advance of those at the other tower as possible. By so doing, the released equipment could be dismantled and transferred to the east bridge while the second set of equipment was completing its work. Sequences in which the units might be lifted and the permissible weights of those units were subject to a large number of variations. Calculations, even by approximate methods, would have required an immense amount of time, and the errors involved would have made it necessary to play ultra safe.

The engineers, therefore, turned over to the contractor one of the models used in the preliminary investigations at the University of California. The model was altered to make it conform to the dimension of the bridge as built and with the truss omitted. The erection was then worked out within the limitations above listed. Later the action of the structure was observed and the actual tower deflections and cable pull checked against the model predictions. The agreement between the two was very close throughout. The same model was later used in establishing the schedule for placing the concrete paving.

The remaining floorbeams and stringers and all other miscellaneous steel was placed by lightweight guy and stiffleg travelers operating on the upper deck of the trusses. The steel was lifted from barges spotted below the point where it was to be placed. At only a few points was it necessary to move any steel along the span. Three of these travelers were employed and completed the erection, except for a few miscellaneous items, between March 16 and June 20.

The connections between adjacent truss units, after they had been suspended from the cables, were made as the joints became fair, with 60 per cent of the holes filled with drift pins and bolts.

Riveting of the chords was permitted only with compression in the joint. Erection of the steel and placing of roadway slabs proceeded so fast that it was difficult to coordinate riveting with either. Over 540,000 field rivets were required.

Paving

Paving of the suspended structure started on June 18, and it is expected that this work will be completed in October. The contractor has a batching and mixing plant in San Francisco between the end of the suspension bridge proper and the anchorage. The mixed concrete is elevated to the upper deck and transferred through a hopper to 3-yd. batch cars. These are hauled to the point of pouring by a gas locomotive.

Deflections of the towers and trusses permit the contractor to pour the center third of the upper deck roadway in a single continuous operation from pier *W*-1 to the center anchorage. A continuous operation on the outside strips and the lower deck strip is not possible because of the resultant torsion of the towers. It is therefore necessary to pour shorter lengths at different points, but in no case is there less than one day's pour available at any point. Generally, several days' work can be continued at each point.

To avoid an excessive unbalanced pull on the center anchorage, about half of the west bridge pavement was placed, after which all the pavement on the east bridge is being poured. Finally, the remaining portions of the west bridge will be completed.

Between pier *W*-1 and the San Francisco anchorage, a continnous truss steel structure has been designed and built. It has two main spans of 378 ft. 1½ in. and a central tower span of 95 ft. 9 in. The truss depth is 38 ft. 3½ in. One of the principal difficulties encountered in this structure was in providing clearance for the suspension bridge cables. At pier *W*-1 the cables are above the trusses but drop past them immediately west of that point. It was impossible to build the trusses inside the cables without restricting the lower deck clearances. They were therefore spaced 75 ft. on centers and the cables passed inside. This required special detailing of several floorbeams to avoid interference.

The spans are continuous over the two supporting bents. Expansion is provided at pier *W*-1 which is the highest end, and the trusses are fixed at the anchorage. Flexure of the supporting bents will accommodate expansion movements at these points.

The suspended structure is part of the superstructure contract for the West Bay crossing, which is held by the Columbia Steel Co. The personnel of the company was published in *Engineering News-Record*, June 20, 1935, p. 867. The design of the erection equipment, including the lifting beams, was made by the erection department of the American Bridge Co. Personnel of the California Toll Bridge Authority was published in *Engineering News-Record*, March 22, 1934, p. 377.

Temperatures in Tremie Concrete

Resistance coil readings on tail race concrete give temperatures at varying depths and for 9 days after placement

By H. A. Moody
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While placing some 300 cu.yd. of concrete in the tail race of one of the hydro electric stations of the Turners Falls Power & Electric Co., observations of internal temperatures of the concrete were made for several days. The results provide new data or the maximum temperatures reached during the period of setting of concrete and the rate of drop in temperature as setting progresses, under a condition where the surrounding temperature is practically constant.

The area of the concrete placed was about 22 x 40 ft., and the depth which was quite uniform was about 9 ft. At a point about in the center of this area, four coils of No. 35 copper wire were secured to a wooden upright extending from the river bottom to above water level which was about 21 ft. above the top of the finished concrete. These coils were located at distances below the top of the concrete of 1 ft., 3.3 ft., 5.7 ft. and 8 ft. Each coil had a resistance of 575 ohms at 72 deg. F. temperature.

The concrete contained 7 bags of cement per cubic yard, the ratio of stone to sand being about 2 to 1 and the slump varying from 6 to 8 inches. It entered the hopper of the tremie pipe at 82 deg. F., while the water temperature was about 32 deg. F. The concrete was placed continually for 12 hr. until the work was completed at 10:30 p.m. on Feb. 16.

Temperature readings, as plotted in Fig. 2, and determined from resistance observations of each coil, were made hourly beginning at 5 p.m. The four curves shown indicate the temperature of the four coils numbered from the bottom to the top, number one being the bottom coil, number four the top. At 5 p.m., the time of the first reading, it is evident from an inspection of Fig. 2 that the concrete had been placed to about elevation of coil number three.

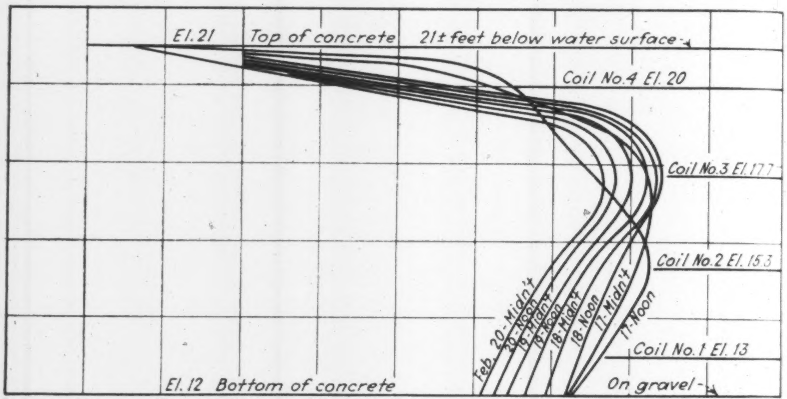
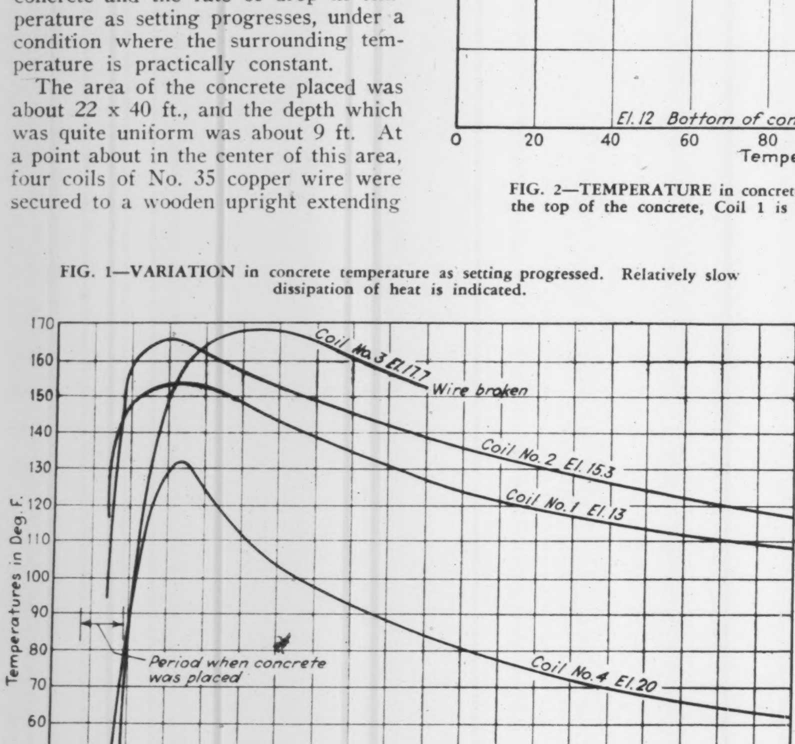


FIG. 2—TEMPERATURE in concrete mass at varying elevations. Coil 4 is 1 ft. below the top of the concrete, Coil 1 is 1 ft. above the bottom or 8 ft. below the top.

FIG. 1—VARIATION in concrete temperature as setting progressed. Relatively slow dissipation of heat is indicated.



From the curves of Fig. 2, the curves shown in Fig. 1 have been plotted to show the probable temperature at all elevations within the mass for each 12-hr. period from midday of Feb. 17 to midnight of Feb. 20.

It would seem futile to attempt to deduce from these observations a law for determining the limit of temperature rise from chemical heat at varying distances from the face of a mass of concrete, or for determining the rate of cooling, as such limits and rates must clearly depend upon the speed at which such a mass is raised to its finished elevation and also must depend upon the heat conductivity of the material surrounding the mass. The determinations, however, should prove useful for estimating the approximate speed of setting of concrete placed under conditions similar to those here described.

Law of Delays and Extras in Government Contracts

A review of the excellent body of law established by the U. S. Court of Claims relating to delays, changes and extras

By **Frederick Schwertner**
Member of the Washington, D. C., Bar, Washington, D. C.

THE COURT OF CLAIMS derives its authority from an act of Congress which provides in part that the Court of Claims shall have jurisdiction to hear and determine "all claims * * * founded upon * * * any contract, express or implied, with the government of the United States * * *." The decisions of this court are reviewable by the United States Supreme Court.

An action in the Court of Claims is commenced by a printed petition, and the same must be filed within six years after the right accrued to avoid the bar of the statute of limitations. A claim founded upon a government contract should not be filed with the court unless it is clearly meritorious under the law, and can be supported by competent proof. As a rule, the principles of law governing contracts between private parties are equally applicable to government contracts.

In this paper I shall deal briefly with the subjects of (1) the deduction of liquidated damages from the contract price because of a contractor's delay in completing the contract; (2) losses sustained by a contractor as a result of delays ascribable to the fault of the government; and (3) changes and extras in the contract work. In the consideration of these branches of the law of government contracts, I shall hereafter quote the pertinent provisions of the standard form of government construction contract.

Liquidated damages for delay

The contract expressly provides that the work shall be commenced as soon as practicable after the date of receipt from the government of notice to proceed, and be completed within a certain number of calendar days after such date. If the contractor fails to complete the contract within the prescribed time, he is required to pay to the government as agreed, fixed and liquidated damages to the amount stipulated in the specifications for each calendar day's delay. The subject of delays is covered by Article 9 of the contract reading as follows:

> Article 9. Delays—Damages—If the contractor refuses or fails to prosecute the work, or any separable part thereof, with such diligence as will insure its completion within the time specified in Article 1, or any extension thereof, or fails to complete said work within such time, the government may, by written notice to the contractor, terminate his right to proceed with the work or such part of the work as to which there has been delay. In such event the government may take over the work and prosecute the same to completion, by contract or otherwise, and the contractor and his sureties shall be liable to the government for any excess cost occasioned the government thereby. If the contractor's right to proceed is so terminated, the government may take possession of and utilize in completing the work such materials, appliances, and plant as may be on the site of the work and necessary therefor. If the government does not terminate the right of the contractor to proceed, the contractor shall continue the work, in which event the actual damages for the delay will be impossible to determine and in lieu thereof the contractor shall pay to the government as fixed, agreed, and liquidated damages for each calendar day of delay until the work is completed or accepted the amount as set forth in the specifications or accompanying papers and the contractor and his sureties shall be liable for the amount thereof: Provided, That the right of the contractor to proceed shall not be terminated or the contractor charged with liquidated damages because of any delays in the completion of the work due to unforeseeable causes beyond the control and without the fault or negligence of the contractor, including, but not restricted to, acts of God, or of the public enemy, acts of the government, acts of another contractor in the performance of a contract with the government, fires, floods, epidemics, quarantine restrictions, strikes, freight embargoes, and unusually severe weather or delays of subcontractors due to such causes, if the contractor shall within ten days from the beginning of any such delay (unless the contracting officer, with the approval of the head of the department or his duly authorized representative, shall grant a further period of time prior to the date of final settlement of the contract) notify the contracting officer in writing of the causes of delay, who shall ascertain the facts and the extent of the delay and extend the time for completing the work when in his judgment the findings or fact justify such an extension, and his findings of fact thereon shall be final and conclusive on the parties hereto, subject only to appeal, within 30 days, by the contractor to the head of the department concerned, whose decision on such appeal as to the facts of delay and the extension of time for completing the work shall be final and conclusive on the parties thereto."

It is perfectly proper for the government to insert in its contracts a provision for the payment of reasonable liquidated damages because of a contractor's delay in completion. The validity of such a clause was sustained by the Supreme Court in *Robinson* vs. *United States* (261 U.S. 486), wherein it was said:

> The provision is not against public policy. The law required that some provision for liquidated damages be inserted. * * * In construction contracts a provision giving liquidated damages for each day's delay is an appropriate means of inducing due performance, or of giving compensation, in case of failure to perform; and courts give it effect in accordance with its terms. * * *

The government usually enforces payment of liquidated damages by a deduction from the final payment due on the contract price. If a contractor has reason to believe that the government improperly deducted liquidated damages, it is advisable for him to accept the final payment under a written protest.

A contractor cannot validly be charged with liquidated damages for a delay which is traceable to some unwarranted act or default of the government. In *United States* vs. *United Engineering & Contracting Co.* (234 U.S. 236), the Supreme Court pertinently remarked:

> * * * We think the better rule is that when the contractor has agreed to do a piece of work within a given time, and the parties have stipulated a fixed sum as liquidated damages, not wholly disproportionate to the loss for each day's delay, in order to enforce such payment the other party must not prevent the performance of the contract within the stipulated time. * * *

If a contractor encounters a delay in the orderly progress of the work by reason of acts or defaults of the government, fires, strikes, unusually severe weather, or other causes within the purview of Article 9 of the contract, he should notify the contracting officer in writing of the causes of the delay within ten days from the beginning therefor. Upon the termination of the delay, the contractor should request a specific extension of the contract time equal to the period of the delay. While the contracting officer may consider an application for additional time which is filed after the ten-day period and prior to final settlement, it is advisable to secure extensions for performance during progress when the delays occur, because the facts and the extent of delay are then more readily ascertainable.

Prompt action needed

As a contractor is relieved from the payment of liquidated damages if he completes the work within the contract period as extended, he should secure during the progress of the work all such extensions of time to which he is properly entitled. A contractor at the time of final settlement receives credit for all extensions granted during the course of construction.

When the contracting officer allows additional time for performance, his decision as to the extent of the delay is binding on both parties, unless the same is modified by the head of the department concerned on an appeal filed by the contractor within thirty days. The finality of such a decision was upheld by the Court of Claims in *Arundel*

Corporation vs. *United States* (79 C.Cls. 343, 362), wherein it was stated:

> It has been held repeatedly by this court and the Supreme Court that where the parties to a contract stipulate as to the finality of the findings of a named official, the same may not be challenged except for fraud or the establishment of such gross error as would warrant an implication of fraud or bad faith. *United States* vs. *Gleason*, 174 U.S. 588; *Penn Bridge Co.* vs. *United States*, 59 C.Cls. 897.

Delays due to government

After a contractor receives from the government a notice to proceed with the contract, he is clearly entitled under the law to commence work without delay and prosecute the same diligently to completion in an orderly and efficient manner without any unwarranted interruption by the government. If a contractor is hindered or retarded in the orderly progress of the work by reason of acts or defaults of the government, he is entitled to be reimbursed for the losses sustained thereby. The case of *Detroit Steel Products Co.* vs. *United States* (62 C.Cls. 686), contains an excellent statement as follows:

> * * * As already suggested, the work was to commence when the plaintiff received notice that the building was ready for it, and, conceding that the plaintiff could not complain of delay in giving the notice, there was yet an obligation on its part to proceed when the notice was given. And there being such an obligation imposed by the contract, was there not a correlative obligation on the government to allow the plaintiff to proceed, without improper interference, once the notice that the building was ready had been given? There have been many cases in this court where the government has been made to respond in damages for delays in or suspensions of or interference with the contractor's work caused by unwarranted acts of the government's agents. * * *

In *Carrol et al.* vs. *United States* (76 C.Cls. 103), it was held that a contractor was entitled to recover damages for delay, where under the contract the government was obligated to clear the site and fix the locations, and the government failed to do so within a reasonable time. The court said (p. 118):

> * * * The contractor, it seems to us, had a lawful right to prepare at once to perform the contract and rely upon the government to comply with its terms, and there is nothing in the record warranting the court to hold that the government possessed the contractual right to delay and retard performance because the officials in charge deemed the contractor's work of less pressing importance than other work going on in the yard. We think the contractual obligation of the government which we cannot ignore, under the quoted paragraphs of the specifications imposed a duty to so act with respect thereto as to enable the plaintiffs to perform their contract work according to the contract, and that the government failed to observe the same. * * *

In *Edge Moor Iron Co.* vs. *United States* (61 C.Cls. 392), dealing with the same subject, the court said:

> The government cannot excuse itself upon the theory that the foundations were not ready, because the delay was caused by the contractors with whom the government had contracted to have them ready. It took upon itself the obligation to have the foundations in place for the performance of the work by the plaintiff; the plaintiffi cannot be punished because of the failure of a third party to perform a contract with which the plaintff had nothing to do.

These decisions clearly support the view that after a contractor receives notice to proceed, the government is not warranted in delaying progress, and should respond in damages for any losses which it causes. While it is the duty of the contractor to perform the contract in a workmanlike manner within the prescribed time, the government has the correlative obligations of permitting due performance.

Suppose a contract provides that the government will furnish plaster models needed in manufacturing architectural terra cotta for the exterior walls of a building, and the government unreasonably delays furnishing the same after request therefor by the contractor, in consequence of which the terra cotta is not available at the job site when needed and the building operation is suspended. In such a case it cannot be disputed that the interruption of the work was due to the fault of the government. It is well settled that if the government undertakes by the terms of the contract to furnish necessary articles and no time is mentioned in the contract, the same must be supplied within a reasonable time after request therefor by the contractor, and failing in this is answerable for damages which flow proximately from its default. As illustrative of this principle, the court in the case of *Donnell-Zane Co.* vs. *United States* (75 C.Cls. 368), stated as follows:

> * * * Final delivery of the required materials was not made until Feb. 7, 1922, practically six months after the time when plaintiff was required to complete the work. During this period the plaintiff was not able at any time to proceed with the work in accordance with any definite plan or schedule, being forced to work as the delayed material arrived, and at times being compelled to suspend the work entirely while waiting for materials.
>
> That the plaintff was delayed in the completion of the work by the failure of the defendant to deliver the necessary materials in accordance with its contract obligations, and incurred increased costs and expenses thereby, is not open to question. The plaintiff is entitled to be reimbursed for the damages so sustained, which the findings fix at $4,621,85. *Detroit Steel Products Co.* vs. *United States*, 62 C.Cls. 686; *McCloskey* vs. *United States*, 66 C.Cls. 105.

In contract cases, the limit of recoverable damages for delay is the additional cost incurred by the contractor which is directly traceable to the government's default, and the amount thereof must be proved with a fair degree of accuracy. The rule as to proof of damages is well stated in *Electric Boat Co.* vs. *United States* (66 C.Cls. 333, 375), where it is said:

> We think the settled rule as to damages is that they are not to be denied because they may not be susceptible to indisputable accuracy. The test to be applied is: Have the sums claimed been calculated upon a reasonable basis, and under all the circumstances of the case does the claimed amount reflect the proximate injury? If the element of speculation is absent, if the basis of computation is the usual and customary one followed in cases of a similar character, and the court is satisfied that the method of computation employed reflects with reasonable certainty the extent of the loss, judgment may be predicated upon this basis. Specially is this true where the loss was occasoned by some act of the defendant. *Eastman Kodak Co. of New York* vs. *Southern Photo Materials Co.*, 273 U.S. 359, 378.

In several cases before the Court of Claims, the government argued that it was not answerable for damages for delay, because it extended the contract time for performance, but the court held that an extension of the contract time does not relieve the government of liability for losses sustained by a contractor, unless the language of the contract forecloses such a claim. In *Levering & Garrigues Co.* vs. *United States* (73 C.Cls. 566), the court made the following pertinent observation on this subject:

> The act of the contracting officer in granting the plaintiff an extension of time in which to complete the contract equal to the delay caused by the government does not relieve the defendant from liability to the plaintiff for losses sustained by it by reason of such delay. *Crook Co.* vs. *United States*, 59 C.Cls. 348; *William Cramp & Sons* vs. *United States*, 41 C.Cls. 164.

In the event a contractor is faced with a government delay which may become serious, he should protest in writing against the same, because under the decisions of the Court of Claims a failure to complain may be treated as a waiver by the contractor of his right to claim damages. It is needless to state that whenever a serious delay occurs, the contractor should cooperate with the government engineer to the end that the causes of delay may be promptly removed.

Changes and extras

If it is determined that changes or extras in the contract work are necessary, and the performance of the same entails additional costs, the contractor should secure written orders from the head of the government department or other authorized representative before proceeding with the same. The contract in so far as material provides as follows:

> **Article 3. Changes—The contracting officer may at any time, by a written order, and without notice to the sureties,**

> make changes in the drawings and (or) specifications of this contract and within the general scope thereof. If such changes cause an increase or decrease in the amount due under this contract, or in the time required for its performance, an equitable adjustment shall be made and the contract shall be modified in writing accordingly. No change involving an estimated increase or decrease of more than Five Hundred Dollars shall be ordered unless approved in writing by the head of the department or his duly authorized representative. Any claim for adjustment under his article must be asserted within ten days from the date the change is ordered : *Provided, however*, That the contracting officer, if he determines that the facts justify such action, may receive and consider, and with the approval of the head of the department or his duly authorized representative, adjust any such claim asserted at any time prior to the date of final settlement of the contract. If the parties fail to agree upon the adjustment to be made the dispute shall be determined as provided in Article 15 hereof. But nothing provided in this article shall excuse the contractor from proceeding with the prosecution of the work so changed.
>
> Article 5. Extras—Except as otherwise herein provided, no charge for any extra work or material will be allowed unless the same has been ordered in writing by the contracting officer and the price stated in such order.
>
> Article 15. Disputes—Except as otherwise specifically provided in this contract, all disputes concerning questions of fact arising under this contract shall be decided by the contracting officer subject to written appeal by the contractor within 30 days to the head of the department concerned or his duly auhorized representative whose decision shall be final and conclusive upon the parties thereto. In the meantime the contractor shall diligently proceed with the work as directed.

If a change or extra is ordered by the government construction engineer at the job site, the contractor should submit a written proposal within ten days from the date the change is ordered setting forth clearly the character of the work to be performed and the additional cost thereof, and secure a written order. If the additional work entails an increase in the time required for completion, the contractor should in submitting his proposal request an extension of the contract time.

As a general rule, the verbal assurances of the government engineer that additional compensation will be allowed for any increase in cost incident to the performance of charges or extras are not binding on the government, and in the absence of a written order, the contractor performs the work at his risk. In *Plumley* vs. *United States* (226 U.S. 545), the Supreme Court succinctly stated:

> There was a total failure to comply with these provisions, and though it may be a hard case, since the court found that the work was in fact extra and of considerable value, yet Plumley cannot recover for that which, though extra, was not ordered by the officer and in the manner required by the contract.

A similar case is *Dale* vs. *United States* (14 C.Cls. 514), where the Court of Claims used this language:

> Where a contract provides that no claim shall be made by the contractor for extra work unless ordered with the approval of the defendant's Chief of Engineers, the contractor cannot recover for extra work or material furnished by direction of the local agent without the knowledge or approval of the Chief of Engineers.

In *Griffiths* vs. *United States* (77 C.Cls. 542), however, it was held that the contractor was entitled to recover the sum of $452, representing the cost of extra grading and filling necessitated by a change in the plans whereby the buildings were turned around and erected in an east and west direction instead of north and south as shown on the original drawings. The court held that the action of the contracting officer in refusing to issue a change order authorizing payment was so capricious and grossly erroneous as to imply bad faith.

In conclusion, it should be emphasized that while the principles of law as applied to government construction contracts are well settled, each case presents its own peculiar problem.

Rubble Jetties Made Solid With Asphaltic Concrete

Hot asphalt mix poured through water penetrates rubble fill and builds up smooth cap under action of vibrators at Galveston jetty

By F. B. Wilby
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DURING AUGUST, 1935, the U. S. Engineer office at Galveston, Texas, was authorized to repair the jetty at the entrance to the Galveston Channel with asphaltic concrete. Before commencing this work extensive research and many experimental tests had been performed under the supervision of the division office, Gulf of Mexico Division. The repair will be completed about Sept. 1, 1936. This application of asphaltic concrete has attracted considerable interest throughout the country and similar work is being contemplated on the Pacfic as well as the Atlantic Coast.

American practice in jetty construction has favored the rubble-mound jetty with or without a foundation mat of fascines or small stones. The rubble mound jetty at Galveston consists of a mound of 75 to 4,000-lb. core stone covered by cap and slope stones weighing 6 to 10 tons, with its top 10 to 15 ft. wide at an elevation of 4 to 6 ft. above mean low gulf and with side slopes of 1 on 1½. (See Fig. 2).

Obviously these jetties are not impervious, and therefore they are subject to the incessant pounding of the seas not only on the exposed surfaces but also in the interior of the mound. During severe storms even the 6- to 10-ton cover stones are frequently displaced. In 1933 a 10-ton stone was lifted and thrown under the lighthouse on the South Jetty at Galveston. The jetties deteriorate in grade and cross-section not only through foundation settlement, but also because of grinding of the core stones on each other and displacement of cover and core stones by wave action during storms. For this reason expensive jetty repairs are periodically required to bring the top up to grade and to consolidate and rebuild the jetty to its designed section.

Shoreward of the low-water line littoral currents carry sand through the pervious jetty into the channel, thus increasing maintenance dredging of the channel. To stop this infiltration of sand through the jetty, efforts have been made during the past five years in the Gulf Division to seal the shore ends of a number of rubble mound jetties at Gulf harbors by constructing concrete caps. These caps have been fairly successful, but due to the difficulty of placing the concrete under water the seal in some cases was not as effective as might be desired.

Asphalt revetment a precedent

During these years the flexible reinforced asphaltic-concrete mat was being developed as a revetment for the banks of the Mississippi River by the Second New Orleans District. This was accomplished economically by combining a mineral aggregate of Mississippi River sand and loess filler with asphalt as the cementing medium. Consideration of the results secured in the experimental work of the Second New Orleans District led to the belief in the office of the division engineer, Gulf of Mexico Division, that the characteristics of asphalt which permitted the production of a flexible concrete mattress might be utilized in producing a mass concrete which could be placed under water to serve not only as a cap to make the jetty impervious to the infiltration of sand, but also as a cementing medium to bind the upper cap and core stones of the jetty into a unified structure more resistant to deterioration than the usual rubble mound. This adaptation of asphaltic concrete for use in jetty repairs involved primarily the problem of producing a hot mix of a consistency which would flow into the interstices of the rock under water, at the same time resisting wave and current

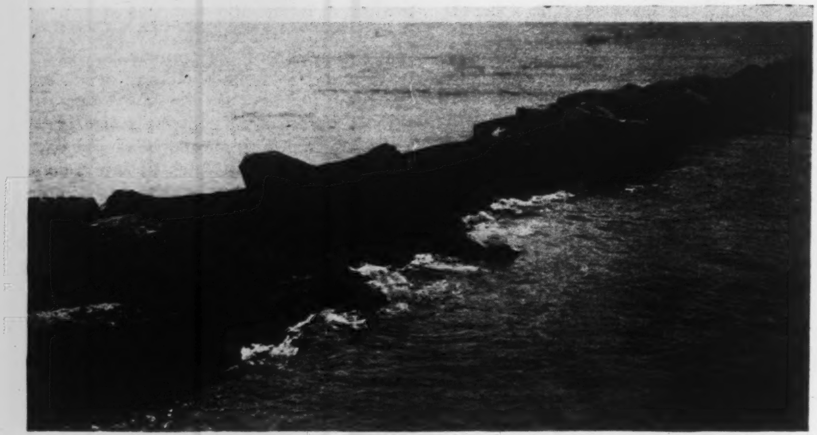


FIG. 1—A BREACH IN THE GALVESTON JETTY which was repaired by filling with hot asphaltic concrete poured through water and settled by vibration.

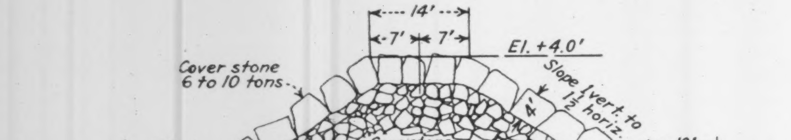
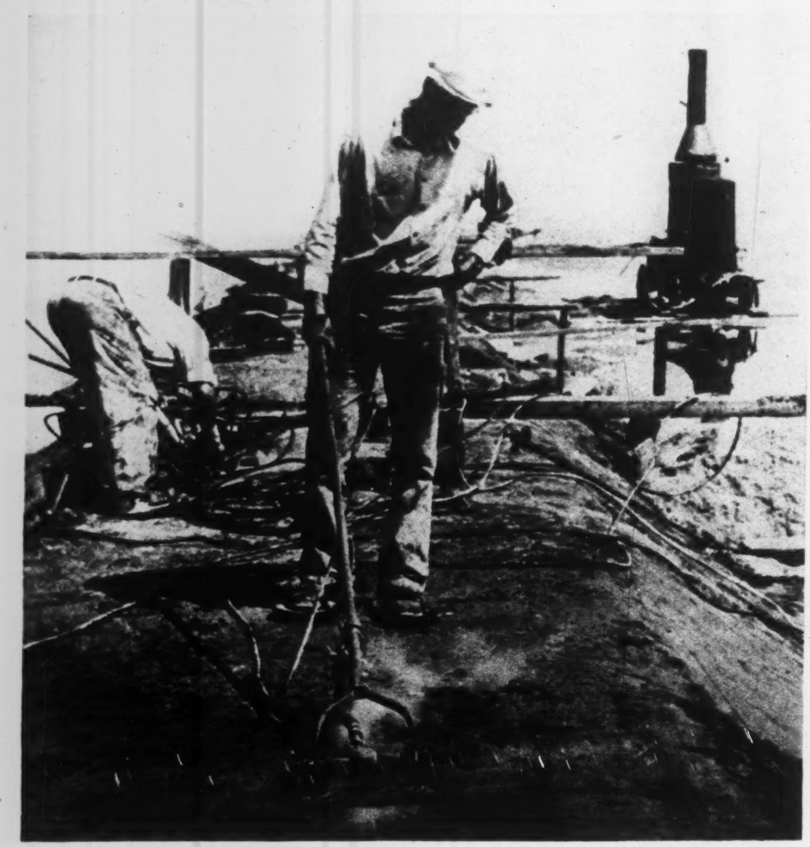


FIG. 2—ORIGINAL SECTION to which Galveston rubble jetty was constructed.

FIG. 3—A SURFACE-TYPE steam vibrator and hand tools are used to build up and shape the top of the jetty after the voids between the stones have been filled.



action of the unconfined sea water, and when cooled would set up as a durable concrete with a permanent bond to the rock above or below water.

Asphalt and cement compared

To test the relative merits of cement concrete and asphaltic concrete, it was proposed that an experimental section of jetting above and below water be constructed in a box on shore at Fort Point, Galveston, that one portion be capped with cement concrete and another portion with asphaltic concrete, and that the box be dismantled for comparison of the two materials. Laboratory tests, directed by F. C. Field of the Asphalt Institute, led to the selection of a "seal course mix" for underwater placement, and a "cap course mix" for that part above water, to insure stability under the summer sun. These were overfilled mixes. Mississippi loess was used as filler and the aggregate was Galveston beach sand.

A watertight wooden box 20x10x2 ft. with a transverse bulkhead at the midpoint, was set up at the Fort Point boatyard. Core stone was placed in the box to a depth of 4 ft. and cap stones were superimposed. Arrangements were made to pump seawater into the box to maintain a depth of 3 ft. of flowing water. Forms 5 ft. square were built over the central portion of the cap stones in each half of the box and extended 2 ft. above the cap stones. Cement concrete was placed as a cap in one form and asphaltic concrete was similarily placed in the other; both materials were thoroughly vibrated. Upon dismantling the box it was found that the cement concrete had failed to set up below the cap stones, although there was about 1 ft. of core stones above the water surface. The bottom course of the asphaltic concrete, however, was a tough durable material that filled the voids in the core stone to a minimum depth of 1 ft. below the water surface. The asphaltic concrete had flowed laterally under the cap rock a maximum of 2½ ft. before it cooled and hardened.

Capping Galveston South Jetty

As a result of this experiment authorization was granted by the Chief of Engineers for capping a portion of the South Jetty at Galveston with asphaltic concrete. An asphalt mixing plant was rented and set up at Fort Point. Power-dumping motor trucks were engaged to transport sand from the beach to Fort Point and to deliver the hot mix to the jetty. Steam-actuated vibrating tools were secured on a hire-purchase plan. A contract was let for asphalt, both 30-40 and 60-70 penetration, to be delivered in tank cars at Fort Point. Arrangements were made to procure Mississippi River loess from New Orleans. On Aug. 28, 1935, the first truckload of 2 tons of seal mix heated to 440 deg. F. was dumped on the jetty into a pool of salt water. With its top surface about 2 in. under water, this mass was vibrated for 30 min. and then a thermometer was inserted. The interior temperature registered 240 deg. A hot mix of asphalt and mineral aggregate had been deposited directly into unconfined water without explosive effects, and the mass had set up as a tough asphaltic concrete.

Truck delivery of asphalt

Dump trucks were used for transporting and depositing the hot mix on the jetty. These trucks moved over the finished asphalt cap and a loose-laid timber runway 300 ft. long which was built above the prescribed top elevation and was moved ahead as the work progressed. The runway was supported on pairs of columns consisting of precast blocks of asphaltic concrete which welded into the cap structure. The hot asphaltic concrete was delivered to the jetty at 400 to 450 deg. F., dumped from the trucks and compacted by steam vibrators. The slopes and top surfaces were formed with shovels and rakes and finished with surfacing vibrators. Forms were not used. The approximate weight of the asphaltic concrete as placed in the South Jetty is 120 lb. per cu.ft.

Experience through several months demonstrated that it is feasible to place asphaltic concrete in sea water and by vibration to cause the material to fill the interstices of the cap and core stone to and well below the water line, that the asphaltic concrete may be compacted by vibration to form a satisfactory surface and side slopes without forms; that asphaltic-concrete jetty capping where sand is available near the jetty should cost less than prevailing costs of cement-concrete capping, and that the rubble stone of the jetty is bonded together by the asphaltic concrete into a solid structure within the section treated. About 3,800 ft. of the South Jetty at Galveston was thus capped with asphaltic concrete upon completaion of the work about July 15, 1936.

Experimenting on jetty repair

As the use of asphaltic concrete in capping jetties had been so successful, authority was obtained to continue the experiments with a view to determining whether the same material would not be of value in rebuilding and repairing offshore sections of jetties where the asphalt would have to be deposited through considerable depths of water. It was contemplated that the experiment should indicate whether or not asphaltic concrete would bond into a solid homogeneous mass if dropped through sea water.

The box at Fort Point was rebuilt and raised to hold a depth of 8 ft. of sea water. Core stone was placed in the jetty to simulate half the section of a breached jetty. Some 22 tons of hot vibrated masses of asphaltic concrete of a modified cap mix in units of 300 lb. were dumped into the box through water, and after the mass had cooled for some days some 16 tons of the seal course mix were dumped on top and vibrated.

These tests demonstrated that masses of hot mix would bond to form a homogeneous compact structure of asphaltic concrete after being dropped through seawater of various depths, and that the compaction of the material by its own weight was greater than had been expected. With respect to the seal mix it was shown that where the material was placed so as to spread out in a thin sheet its heat was soon lost and considerable waste resulted. It was suggested that a new mix be designed and that a further test be made.

The box was cleaned out and rebuilt, and again core stone was placed to form an irregular bottom simulating a breached jetty. A new mix was designed to be placed at about 400 deg. F. with a clamshell bucket. This mix consisted of 18 per cent of 30-40 penetration asphalt, 12 per cent of loess and 70 per cent of Galveston beach sand. About 38 tons of hot mix was placed in the box of water to simulate a restored section of half of a jetty about 13 ft. high, extending about 5 ft. above the water line. The top portion of the mass was vibrated. The box was torn down and air chisels were used to cut a trench through the mass. Here too it was shown that the hot mix dropped through seawater had set up as a tough homogeneous asphaltic concrete.

Breached jetty repaired

Actual placement of hot-mix asphaltic concrete to restore offshore breached sections of the South Jetty at Galveston was undertaken in March, 1936. The material was loaded into an insulated steel barge at Fort Point and towed through the jetty channel to the breached section near the seaward end of the jetty. There a derrick boat equipped with a clamshell lifted the hot-mix from the insulated barge and dropped it into the sea to fill the gaps. Vibrators were used to compact the upper portion of the mass, but dependence was placed on the weight of the superimposed material to compact the lower portion of the asphaltic fill.

The use of asphaltic concrete in jetty construction is now in the development rather than the experimental stage. It has been demonstrated that a suitable hot overfilled bituminous mix may be dropped into unconfined seawater to form a solid homogeneous asphaltic concrete, and similarly that a suitable hot mix may be placed on a rubble-mound jetty to fill the interstices above and for some distance below the water line and this consolidates the rubble stones into a solid structure where the interstices are filled with asphaltic concrete.

The experimental work and repair work on the jetties at Galveston has been performed by the U. S. Engineer Office at Galveston under the supervision of Col. E. H. Marks, district engineer, Col. Warren T. Hannum, and his successor Col. F. B. Wilby, division engineers, Gulf of Mexico division. The engineers directly in charge of field work are Capt. C. F. Baish, Robert B. Tinsley, and James A. Cotton. Especial credit is due to Capt. P. H. Timothy and to R. M. McCrone of the division office, and to F. C. Field of the Asphalt Institute, for developing the methods and materials in use.

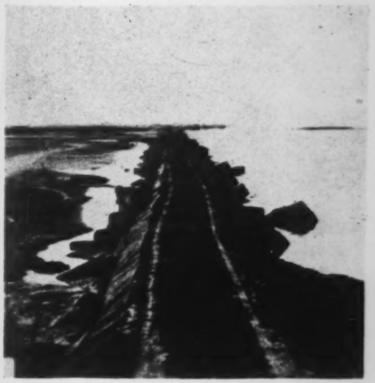
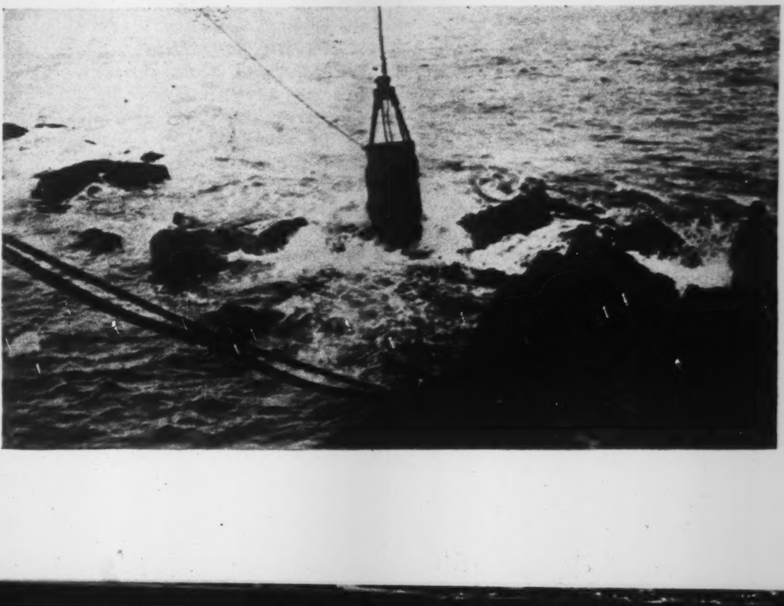


FIG. 4—COMPLETED SECTION of South Jetty at Galveston, Tex., showing appearance of asphaltic concrete.

FIG. 5—REPAIRING A BREACH in a rubble jetty by depositing hot asphaltic concrete onto the rubble through water; a repaired breach is shown by Fig. 1.



MOSQUITO CONTROL ENGINEERING IV

Inland Control Methods

Drainage of swamp areas and rendering water unfit for breeding when certain mosquito species are prevalent constitute the basic elements of control. The structures and facilities of modern life in large centers of population complicate efforts directed toward eradication.

By J. Lyell Clarke
Sanitary Engineer, Des Plaines Valley Mosquito Abatement District, Riverside, Ill.

MOSQUITO control activity in the United States has resolved itself into three well-defined divisions concerned with: Salt-marsh varieties; malaria-carrying mosquitoes; and inland fresh-water species. These classifications have not been chosen arbitrarily but are the outgrowth of field experience based on the habits and geographical location of certain kinds of mosquitoes. While this article will be confined to a discussion of the so-called inland group and will review eradication measures that have been found most effective in coping with this phase of mosquito abatement, the basic elements of mosquito control are of wide general application.

Perhaps the most practical and logical manner of presentation is to adopt the cyclic order of the seasons, spring, summer and autumn, because definite groups of mosquitoes with similar breeding and biting habits are aligned with climatic conditions represented by these three periods of the year. In order to further simplify the discussion, one species that has attained economic importance in each seasonal group has been chosen to represent in general, the characteristics of that group.

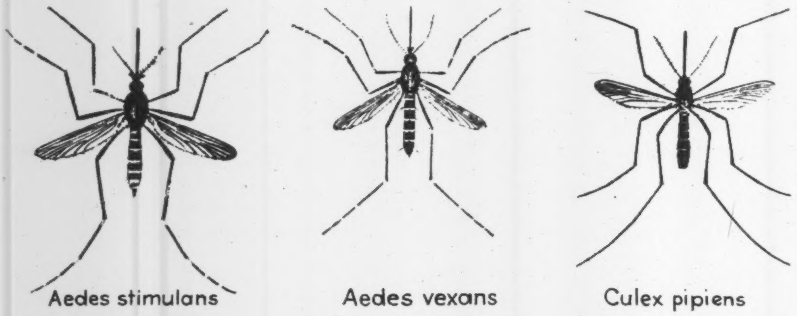


FIG. 1—TYPICAL INLAND SPECIES of mosquitoes. From left to right: *Aedes stimulans*, which appears in the spring; *Aedes vexans*, a mid-summer pest; and *Culex pipiens*, whose numbers reach a maximum by early autumn

Spring mosquitoes

In the month of March when pools form from the melting snow and several warm days come in succession, the temperature of shallow water rises high enough to hatch the eggs of the spring group of mosquitoes. The very first hatching occurs in the woodland pools, as here the water is protected from the cold wind and there is no foliage on the trees to obscure the rays of the sun.

The species of mosquito most representative of this group is the *Aedes stimulans* shown in Fig. 1. It is quite large with a brown, longitudinal stripe on its back. The upper surface of the abdomen under the wings is banded with white and the legs also carry wide white bands. Only one brood is produced each year. The eggs which are laid by this early brood lie dormant until the following spring.

Species of the Spring Group—The most prevalent mosquitoes occurring in the spring season are: *Aedes excrucians, Aedes stimulans, Aedes fitchii, Aedes punctors,* and *Aedes cinereus.* The last two may give annoyance in some localities on the outskirts of populated areas or in woodland parks in villages, but they are not so common over a wide range of territory as the first three.

In passing, it might be said that in dealing with this group of mosquitoes one is apt to infringe upon the much-coveted privilege of the conservationists. Fortunately, this group of mosquitoes produces only one brood, its breeding grounds are restricted to well defined small areas, and these may be sprayed with pyrethrum larvacide which does not harm vegetation.

Control Measures—It is advisable to start the control campaign before actual breeding begins since drainage structures always require cleaning after the winter season. Immediate attention should be given to woodland ditches, but streams and large marshes may be ignored because they still contain cold water. Woodland pools are the first to produce mosquitoes, but inasmuch as the mosquito larvæ which appear in April are single-brooded, drainage systems in the woodland areas need not be too extensive. Only the large pools need be connected with the ditching system or farm-tile drains, leaving small depressions of shallow water to supply moisture to trees and shrubbery. Shallow pools may be left to dry up or they may be sprayed with pyrethum larvicide at a nominal cost.

The labor force required for early clearing and spraying of woodland pools is usually about one-twentieth of that necessary during the remainder of the season. This early spraying is very important, however, in order to destroy larvæ which slowly hatch in the melting snow pools to reach the adult stage in about 30 days.

Inspection of suspected breeding areas should be made only on warm, sunshiny days. On cold days the larvæ will be hidden beneath the leaves and in the mud at the bottom of the pools. While many inspections in April may prove futile it is quite likely that during the latter part of the month and in the first week of May, large lavæ or pupæ will suddenly appear.

This program for eradication applies to all mosquitoes in the early group. As only small areas, widely scattered, are involved in spring operation, the equipment best suited for this work is a truck to transport the men, mosquito larvacide, and a supply of 1½ gallon spray cans (Fig. 2).

In June and July when long, wet spells come with the greatest frequency and the water which floods the bottom lands, marshes and field pools is very

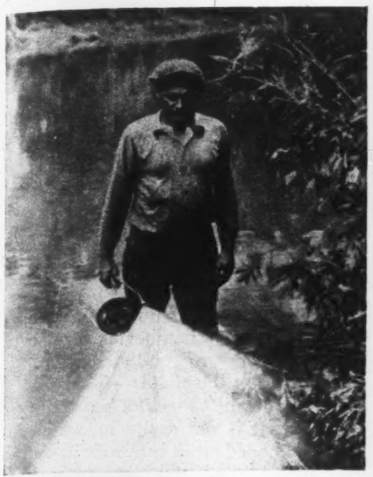


FIG. 2—PORTABLE SPRAY CANS, holding 1½ gal. of larvicide, are used for intermittent spraying operations

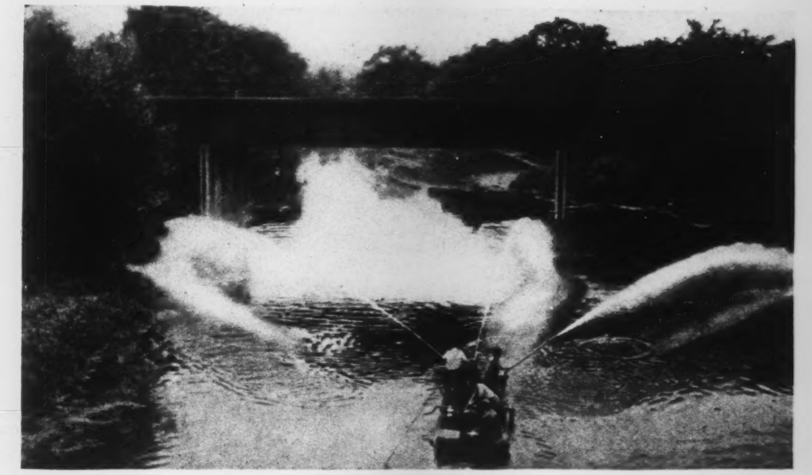


FIG. 3—SPRAY SCOW used by the Des Plaines Valley (Ill.) Mosquito Abatement District to obtain speedy and effective larvicidal coverage over an entire river surface

warm, conditions are at their best for the production of hordes of flood water mosquitoes.

The species which best typifies this group is the *Aedes vexans* shown in Fig. 1. It is a medium-sized mosquito, often considered small. It has a pointed abdomen, a white band constricted at the center to form a white "B" at the abdominal segments, and legs with narrow white bands.

The adult of this species does not survive the winter. In order to perpetuate its race it deposits eggs which remain viable under all climatic conditions, withstanding the coldest winter and hottest summer. It deposits eggs in depressions like the early spring varieties, although it differs from the latter in that it produces many broods. As an annoyance factor the very early broods are not important, but the eggs deposited by these broods may add to the number of hatchable eggs later in the same season. In the summer large broods occur more often because of the frequency of wet spells; occasionally a rainy September will produce a brood equal to that of midsummer.

The eggs of the *Aedes vexans* are deposited in grassy wet lot pools, flooded subdivisions, casual field pools, along the marginal edges of permanent marshes (particularly those which are flooded intermittently), and in prodigious numbers in the flood plain back pools. This knowing little insect can discriminate between those places where water will collect and those where it will not. You wonder how it knows. I surmise that its discrimination is based upon accurate knowledge of water grasses or other indications which we have not as yet learned to recognize.

Sudden infestations usually occur twenty days after a wet spell. The flight range of this species and some others of this group may be ten miles. Recently, staining methods have been developed in an attempt to trace its flight habits more accurately.

Summer Group Species—While *Aedes vexans* is the most widely distributed of the summer flood pool mosquitoes there are perhaps twenty other species which have to be reckoned with following long wet spells. *Aedes trivittitus* breeds in field pools but flies to woodland areas where it remains. In the Northwest, *Aedes aldrichi* may reign supreme. *Aedes punctors* is another woods mosquito. *Aedes campestris* is the mosquito of the plains. In Utah special problems arise from flooding of land by irrigation dtches producing continuous breeding of *Aedes dorsalis.*

In addition to these typical flood water mosquitoes, there are several species of the genus, *Psorophora,* which develop very rapidly in transient field pools within 4 or 5 days. One of these is the largest species in the United States, *Psorophora ciliata.* Its name translated means a hairy itch producer. *Psorophora ferox,* a large purple mosquito with white-tipped hind legs, is noted for the pain it is capable of inflicting.

The *Mansonia perturbans,* a house-entering species is quite widely distributed over the United States, although this distribution is spotty in character due to its highly restricted breeding habits. It is a severe biter and where it does occur in abundance it is a literal scourge. It is likely to occur in permanent marshes having a peaty bottom where the larvæ attach themselves to pickerel weed, arrowhead and other succulent plants. In Illinois this species is single-brooded, adults appearing during the first week in July. In Florida, however, where there is a longer breeding season, two broods are produced—one in May and another in August. Its life cycle is so unlike that of other mosquitoes that control measures ordinarily employed against other species are ineffectual. For this reason special reference is made to the work done by T. E. McNeel, Bureau of Entomology, U. S. Department of Agriculture, appearing in the *Proceedings* of the Nineteenth Annual Meeting of the New Jersey Mosquito Extermination Association, 1932, p. 91, New Brunswick, N. J.

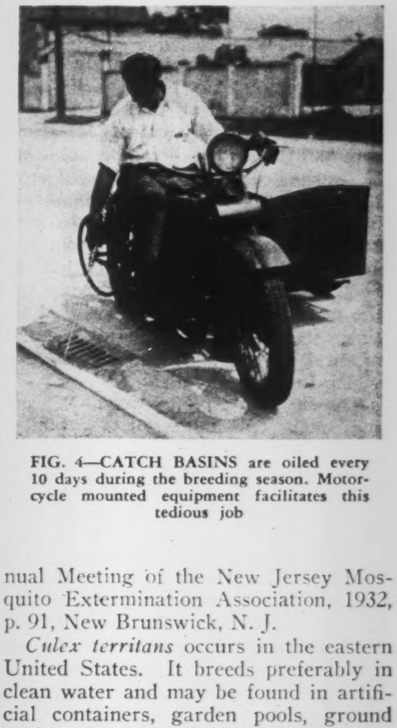


FIG. 4—CATCH BASINS are oiled every 10 days during the breeding season. Motorcycle mounted equipment facilitates this tedious job

Culex territans occurs in the eastern United States. It breeds preferably in clean water and may be found in artificial containers, garden pools, ground pools and roadside ditches. In the northern and western states it appears earlier than *Culex pipiens* and disappears as pipiens reaches its peak of production in August.

Control measures

In dealing with mosquitoes that are in this group I wish to emphasize the fact that drainage is the basic element of control. Drainage must be relied upon to pare down the problem to a point where spraying of temporary or casual pools becomes a very small part of each season's work. Until an adequate drainage system is installed flood conditions may tax any mosquito abatement worker to the limit.

Application of larvicide is not a fool-

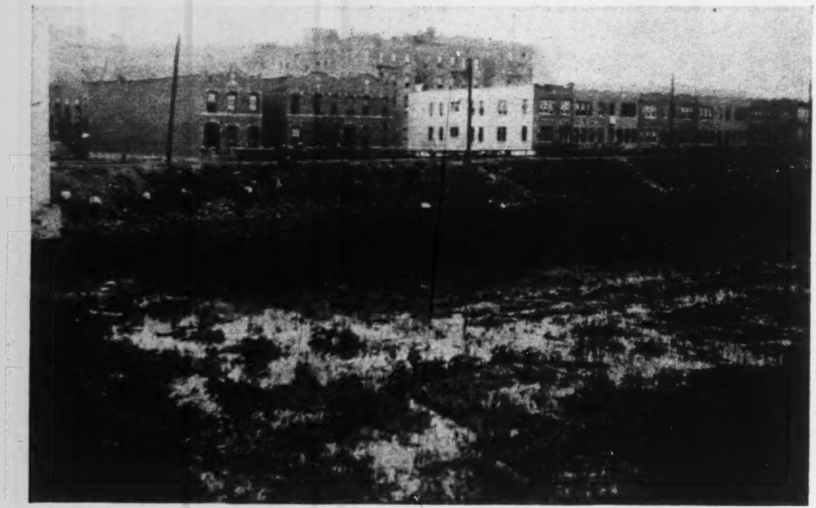


FIG. 5—MOSQUITOES make no distinction between man-made marshes and their native swamp abodes. Prolific breeding occurs in the wet lots that fringe on the edges of built-up territory

proof measure—it is only supplementary to drainage, and not to be wholly relied upon. In spite of all that can be done, even in the most efficient organizations, here and there small breeding areas will be overlooked or not sprayed uniformly. Large marches should be drained, making it necessary to spray only the small casual pools, ditches and bottom land pools where larvæ are left when the flood waters recede. At times like this, the maintenance force is increased five times its normal strength for a period of ten days, with the regular men supervising the new recruits.

During such peaks of potential breeding, small pools and wet lots in built-up territory receive first attention. Small pick-up trucks manned with a foreman and four sprayers comb the villages to locate and spray flooded lots. Concurrently, large trucks with a big crew are dispatched to spray the undrained marches. By the time this work is finished the flood will have receded, leaving small pools of concentrated larvæ in the flooded bottom lands and in the slow eddies along the main streams. Some of the crew are then dispatched along the streams while others scatter over the flood plain in search of small entrapped water areas.

If the streams are deep enough power sprayers are placed in boats to apply larvicide on the stream and backwater pools immediately adjoining the main stream. The barge is driven downstream with an outboard motor. As a rule the more prolific breeding areas are too far removed from roads to permit of anything but a large crew of men equipped with hand sprayers followed by supply men carrying five-gallon cans of concentrated pyrethrum larvicide which is diluted 1 to 10 with water from the marsh to be sprayed. All wet lots, marshes and other breeding places are recorded each season to be included in the next season's drainage program.

Trucks equipped with high pressure sprayers similar to those used in spraying trees may be brought into use in flooded regions or in areas bounded by paved streets. Nevertheless, the brunt of the attack must be carried by many trucks, many men and many small hand sprayers that can be quickly maneuvered into place, there being only five to eight days elapsing between the hatching of myriads of eggs and emergence of the winged insect.

After the campaign to spray breeding places is over, the extra help is laid off and the regular crew resumes its normal routine of maintenance work and drainage improvement.

Advancing the drainage program

Often where large breeding areas are involved, it is advisable not to attempt control of the summer brood by spraying their breeding grounds. The drainage program should be well advanced before spraying can be relied upon. The marshes to receive first consideration are prairie or open field marshes which are intermittently wet and dry. Next for attention should be the low-lying depressions in the flood plain of rivers and streams. After completion of the work on the large and prolific breeding areas a program may be started for the drainage of wet lots in villages.

Permanent Swamps—Permanent marshes are not necessarily mosquito breeders. On the other hand, permanent marshes are very important places of refuge for wild life. Fish control of mosquito breeding should be relied upon in those special cases where it is desired to preserve bodies of water which are important as the feeding and nesting grounds of birds, or where they have an inherent appeal to the nature lover who appreciates them as part and parcel of the native landscape. Such marshes may be made mosquito-proof by deepening of the central area in each pond and constructing radial connecting channels to other low spots in order to provide passageway for fish. In like manner, deep channels dug from rivers to connect with back pools will serve as passageways for the ingress and egress of fish from the main streams into the bottom-land pools. The small top feeding minnow, *Gambusia affinis,* is a particularly good control agent, and it becomes acclimated to northern regions. The dwarf sunfish, *Eupomotis,* is native to the northern region and may prove a valuable adjunct in mosquito control work wherever hatcheries are established to permit of its distribution.

Subdivision Drainage—There is another important source of mosquito production that is not easily recognized. I refer to flooded block interiors in outlying subdivisions and the remnants thereof which remain as wet lots in built-up territory. In the spring and summer, water is impounded by elevated sidewalks, and mosquitoes, who make no distinction between these man-made marshes and their native marsh, breed prolifically.

The municipal engineer would do well in requiring real estate subdividers to properly drain these areas before permitting the property to become a part of the villages.

Box Culverts—When I mention the box culvert, be it on highway or railroad, I speak of a structure that is highly conducive to mosquito breeding; it is found in built-up districts to some extent, but is particularly evident in the outlying countryside. This is a distinct public menace in so far as mosquito production is concerned, not because of the small breeding pool which occurs at the upstream end (since the engineer may be pardoned for placing the flow line of the culvert just a little above the streambed in order to prevent silting and thereby heavy maintenance costs) but the unpardonable dereliction in the design of a bottom which, being an integral part of the structure, may not be tampered with. This untouchable bottom in 90 per cent of the cases not only deprives the upstream private landowner of his agricultural drainage rights, but in many instances prevents drainage of large upstream mosquito breeding swamps. This is a practice which some conscientious highway engineer, having the interest of the public at heart, will set aside by substituting a bottomless culvert which will allow the deepening of the streambed. In instances where high box culverts block the drainage outlet of upstream property owners, the only recourse is to jack a corrugated culvert under the structure.

Culvert criticism would not be complete if I omitted mention of the culvert known as the "equalizer"; *i.e.,* a culvert which has no outlet but simply permits water to stand at the same level on both sides of the road. Its use in highway construction is an admission of failure. Standing water in borrow pits and roadside marshes should be drained as an integral part of highway construction.

There is no good reason why land should not be purchased or condemned for drainage rights-of-way in the same manner employed to obtain road rights-of-way.

Autumn mosquitoes

When the middle of July arrives and the streams reach a low water stage, another type of mosquito makes it appearance—*Culex pipiens* in the northern states, *Culex quinquifaciatus* in the southern states. Both are alike in habits, preferring polluted water. Both have a very detestable habit of entering dwellings, which is not done by the woodland and flood water mosquitoes. The female hibernates during the winter in underground hideouts, and in April, the first raft of eggs is deposited on standing water. From this time on brood follows brood by strict geometric progression, until in August the number on the wing has reached large proportions.

The *Culex pipiens* (Fig. 1) is a dull grey mosquito of medium size, having a blunt abdomen, white bands at the abdominal segments, and uniformly dark legs. Its larvæ are found in barrels, tubs, street catch basins and polluted streams. Unlike the spring mosquito inhabiting woodland pools, and the summer mosquitoes inhabiting flood water pools, this domestic autumn species is found close to human habitation.

Autumn Species—The *Culex salinarius* is found in the Mississippi watershed as well as in the eastern and southern states. It breeds in brackish marshes along the seacoast and inland in polluted marshes. The *Anopheles punctipennis* is found in the slack waters of clear hilly streams as well as in large marshes in the upland regions. *Anopheles quadrimaculatus*, the true malaria carrier, is a pond breeder found in the lowland marshes and bayous. Both *punctipennis* and *quadrimaculatus* are found in garden pools, the *punctipennis* predominating.

Breeding Places—The polluted stream is the most important and prolific source of mosquito supply. Sewage pollution depletes the oxygen in water, destroys fish and other aquatic enemies of the mosquito and makes of such waters literal incubators of mosquito production. A sewage polluted zone in a stream disturbs the biological balance and is not only a death trap for migrating fish but brings about increased mosquito production over the entire watershed.

Next in importance is the street catch basin. Open catch basins with the water directly beneath the grates constitute ideal breeding places throughout the summer season. The closed type, where the inlet is at the gutter and the collecting basin type used in parkways, do not encourage mosquito breeding, because the insect has to pass through the inlet pipe to find water. This type of basin, however, is more apt to harbor mosquitoes during the winter months, as it affords better protection for the hibernating female. It might be well to point out at this point that catch basins are a useless heritage of the past imposed upon modern civilization. Where sewer grades will permit a velocity of 2½ ft. per sec., catch basins should be eliminated. Authorities on the subject of sewer design substantiate these claims.

Another source of mosquito production of lesser importance in the modern city is that which takes place on private property in barrels, tin cans and other water containers. Mosquito production in such places never assumes proportions where it affects large areas, but it may be the cause of acute local annoyance.

The garden fish pool, as a mosquito hazard, is viewed with alarm by many. In reality a properly tended garden pool is a safety valve in any community in that it is an actual depository for mosquito eggs from which mosquitoes never emerge. It is my opinion that a properly tended garden pool is truly a mosquito trap. The humidity, shade, and foliage around a garden pool offers every inducement as a harboring place for the adult mosquito and the water is a fine place for egg-laying. After the female lays an egg raft on the water surface, the larvæ will develop only to be devoured by the fish. Were it not for garden pools the female mosquito would be forced to find some other body of water, perhaps a hidden source not so likely to be found by the owner or the inspector.

Control costs

Control Measures—In the latter part of July a rainfall of one-half inch or more may flush mosquito larvæ from street catch basins into streams or river, and in the absence of fish to devour the larvæ, the stream quickly becomes alive with mosquito wrigglers. Conditions such as this require prompt action. Power spraying equipment mounted on scows (Fig. 3) must be pressed into service at once to apply larvicide to the entire river surface.

Small streams are patrolled by a crew using hand sprayers. These efforts are continued in a routine manner until cooler weather. This work is supplemented by spraying of street catch basins with hand sprayers or preferably by the use of motorcycle mounted equipment (Fig. 4). Furnace oil is used as a spraying material, application being made to the street catch basins once in 10 days, or, when wood tar is used, applications are extended to a period of one month.

Inspection of home premises plays a major part in the control of this species because the source of much annoyance, locally, is often traceable to barrels, tubs and other water containers. Special attention should be given to manufacturing districts where fire protection barrels are in use, and to villages having inadequate garbage collection systems.

The cost of mosquito control, depending upon the population density in the area under consideration, ranges from 12c. to 25c. per capita. The cost of primary drainage of swamps, borrow pits, wet lots and low areas in flood plains will average about $2,500 per sq.mi.

The propagation method of blasting ditches with dynamite may be relied upon as a cheap and efficient method of draining large swamps located more than one-half mile from populated areas. By this method one mile of ditch may be constructed in one day with a labor force of five men. The most favorable time is when the ground is water soaked. The cost is about one-third that of hand labor.

While the correction of stream pollution is not a function of mosquito abatement, nevertheless, one-half of the cost of mosquito control in inland villages is chargeable to routine inspection and spraying of polluted streams. This being the case, a great deal of educational and promotional work is warranted among municipalities to bring about the reclamation of polluted streams.

As a rule it will be found that mosquito breeding grounds constitute only about 3 per cent of adjacent land area. Water areas, however, constitute a much higher percentage but fortunately mosquitoes do not breed in all water. Clean rivers, streams, ponds, and quarry holes do not produce mosquitoes under normal conditions. If, under apparently normal circumstances, such places are found to produce mosquitoes, expert advice should be obtained.

Salt marsh mosquito control will be discussed next week by Ralph J. Van Derwerker, superintendent of the Union County, (N. J.) Mosquito Extermination Commission.

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The Engineering Foundation Organizes Committees For Welding Research

The subcommittee on industrial research of the Engineering Foundation welding research committee, at a meeting on July 23-24, organized a group of subcommittees to conduct a program of research on welding problems.

Six subcommittees on materials were formed, to study: cast iron, chairman not yet named; carbon steel, J. C. Hodge, chairman; low alloy steels, J. H. Pritchett, chairman; high alloy steels, C. H. Nelson, chairman; aluminum alloys, G. O. Hoglund, chairman; copper alloys, D. K. Crampton, chairman; nickel alloys, O. B. J. Fraser, chairman. Three functional subcommittees are being organized: methods of testing, M. F. Fayre, chairman; analysis of weld failures, chairman not yet named; weld stresses, E. Chapman, chairman.

The Engineering Foundation has made three grants, totalling $12,000, to launch the project.

Soil Mechanics Notes—II

Summaries of important practical information extracted from selected papers presented at the International Conference on Soil Mechanics and Foundation Engineering

SELECTED EXTRACTS from the papers presented at the international soil mechanics conference, the first group of which was published in the issue of July 23, is continued here with examples of soil solidification and foundation practices of notable interest. The 160 papers (See *ENR* July 2, p. 21, July 9, p. 54; July 23, p. 114) considered at the conference presented a wealth of useful records of research and experience, only a few of which can be mentioned. The following are summaries of selected papers in addition to those summarized in our issue of July 23.

Impermeabilizing with bitumen

A process of making pervious subsoils impermeable and coherent by injecting an asphalt emulsion was described by J. Ph. Pfeiffer of the Antwerp laboratories of the Royal Dutch-Shell oil group. The distinctive feature of the process is that it is effective in very fine material; in the two practical examples described 90 per cent and 82 per cent respectively of the particles passed the 40 screen. In both operations complete impermeability was obtained.

The dilution of emulsion used is practically as thin as water so that under moderate pressure in the jet pipes it can be forced into fine soils replacing the ground water. The agglomeration of the bitumen particles is induced by coagulants which are added to the emulsion and which can be composed to act at any determined time from a few minutes to several days. Incidentally a process has been developed recently by which cement grout can be combined with the asphalt emulsion to make a solidifying agent for coarse soils and fissured rock.

As an example of practical use, Fig. 1 shows the method of sealing a foundation pit for a bridge pier at Gouda in the Netherlands. In driving concrete foundation piles inside the sheetpile cofferdam the clay layer was penetrated and water welled up and filled the pit. In the pit area, about 22x75 ft., 60 jet pipes were sunk into the sand to a level just below the bottom of the sheeting and the whole area was injected and made watertight with the bitumen emulsion. (The process described is apparently proprietory.—Editor.)

Electrochemical hardening of clay

A promising process of hardening clay by electrochemical action was outlined in a paper by K. Endell and U. Hoffmann, Technische Hochschule, Berlin. In this process metal electrodes are introduced into the soil at "appropriate spacings"; aluminum is always used for the anode while the cathode is usually copper. Direct current at an "appropriate voltage" is allowed to pass between the electrodes until the clay is hardened.

Various laboratory tests are cited. In one a clay with 80 per cent water content (almost slurry) after "treatment for some time" with a current of 8-14 amperes at 300-500 volts became so hard that an iron rod with an end of 1 sq.cm. (0.155 sq.in.) failed to penetrate the sample under a load of 22 lb. Specimens cut from the hardened sample close to the electrode neither disintegrated nor swelled during several months immersion in water. Atterberg's liquid limit and the angle of internal friction are increased by treatment. Explanation of the complex action that occurs and some further test data and conclusions are given by the paper as follows:

> This hardening is apparently brought about by a change in the electrical charge on the clay particles, caused by the passage of the electric current through the clay. According to our present picture of this complicated process, this change of charge gives rise to an exchange of the exchangeable kations bound on the surface of the clay particles, particularly of *Na* against *H* or certain trivalent ions, particularly *Al*. The *H* is derived from the electrolysis of the water in the clay, and the aluminum is supplied by the A1-anode.

No matter how large the water content

FIG. 1—INJECTION of asphaltic emulsion seals a foundation pit for a bridge pier in Holland; a timed coagulant causes the bitumen to agglomerate in water.

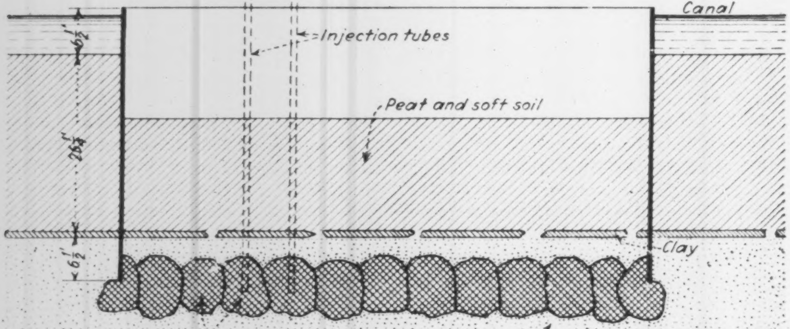
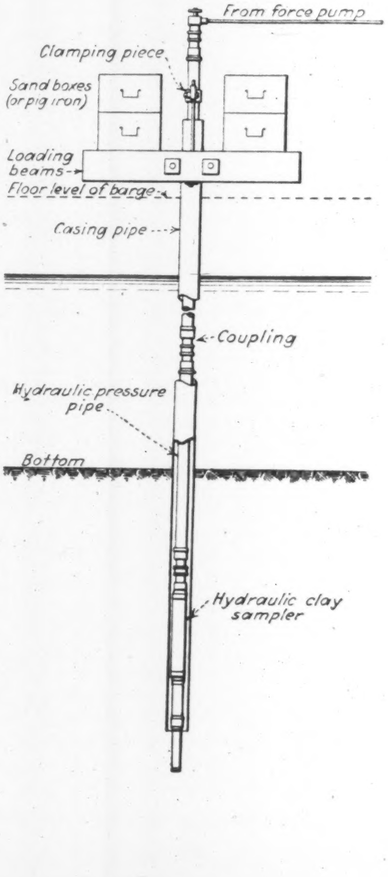


FIG. 2—DEEP SAMPLING ARRANGEMENT at Storstrom bridge using a special hydraulic sampler.



of the soil was before treatment, it sinks to 30 or 40 per cent afterwards, the exact amount depending on the nature of the soil, and the hardening is of such a nature that even in the presence of excess water, the water content of the treated material remains constant. The extraordinary small compressibility of the treated soil suggests that the hardening process goes hand in hand with the development of some sort of structure, even if the original material was thoroughly kneaded before treatment. Apparently the influence of the electric current is such that a disturbed soil regains the properties of an undisturbed one after a short period of treatment. In addition to the change in the charge on the clay particles discussed above, it is possible that deposits of aluminum compounds in the clay may contribute to the strength of the treated material. It may be that aluminates are formed which have an effect similar to that of the calcium-aluminate-hydrate produced during the setting of cement. The white precipitate on the aluminum anode has been determined as bauxite ($Al_2O_2.3H_2O$) by X-ray methods.

Dr. Erlenbach has recently carried out interesting laboratory experiments with 60-cm (24-in.) model piles, whose lower half was covered with aluminum or copper sheeting, respectively. Using these piles as electrodes, with a potential of 14 volts and a current of less than 0.1 ampere per pile, Erlenbach was able to raise the bearing capacity of the soil from 21 lb. per sq.in. to 100 lb. after a five hours' treatment, to 206 lb. after 20 hours, and shortly thereafter to 242 lb. per sq.in. The force necessary to pull the piles was about the same as the bearing capacity of the piles. Around the sheet-metal covering of the piles was a cylinder of hardened clay which adhered to the piles when they were pulled.

The results of the experiments just described and other recent contributions to our knowledge of the electrochemical nature of clays supplement one another perfectly. It is of course too early to say whether or not the new process may be applied economically under all circumstances to all clay soils. The question of the most economical size and arrangement of the electrodes, on which the reliability of the process depends, is by no means settled.

It is however worth mentioning that this process represents the first attempt to harden clay soils for engineering purposes. Since the hardening process itself rests on a sound theoretical basis, there are excellent prospects for a successful application of the method in all cases of foundation and mining engineering in which a horizon of soft clay is encountered. (The process has been patented by L. Casagrande of Berlin.—Editor.)

Foundations for Storstrom Bridge

Foundation studies for the Storstrom bridge introducing a demountable cofferdam; a new device for taking undisturbed soil samples; a cone penetrometer apparatus for laboratory tests, and a device for measuring the stresses in sheetpiling were described in a paper by A. E. Bretting, chief engineer, Christiani & Neilsen, Copenhagen, Denmark. Briefly the Storstrom bridge providing railway and highway connection between the islands of Zealand and Falster is about 10,500 ft. long and is carried on 51 piers. The paper relates only to the substructure which is now about completed.

At the crossing the average depth of water is about 26½ ft. and the piers are founded on clay generally at a depth below bottom of 6½ to 10 ft. The maximum foundation depth is 52½ ft. below water level. The clay is glacial of variable consistency and under it is deep chalk. Two methods of construction were employed depending upon the consistency of the clay; in stiff clay a cofferdam was used and the pier base was concreted dry but in less resistant clay the pier base was concreted under water; in both cases the pier caisson was floated onto the base and the pier was concreted in the dry. Only the first method used on 24 piers is described since the cofferdamming practice was unique and because the work was very rapid—a pier completed to 10 ft. above water surface in less than a month.

Demountable Cofferdams—A ring cofferdam, Fig. 5, is floated over the pier site and seated on short piles. Around the outer periphery of this coffer is hung in advance a ring of sheet piles interlocked and ready to drive. When the coffer is seated the sheetpiles are driven by an underwater hammer until the tops are just above the lower edge of the coffer and the space between it and the piles is calked by a hemp rope continuous around the pier. When the cofferdam is pumped out the water pressure automatically tightens the rope seal. Excavation and concreting proceed in the dry, the sheetpile ring and the inside shell of the floating unit serving as forms for the pier base and the lower part of the pier shaft. This is concreted only to 10 ft. below water and then the ring dam is unballasted and floated clear to be reused for another pier; the sheetpile ring is left in place. A reinforced concrete caisson floated from shore and set on the completed base part forms the pier from 10 ft. below to 10 ft. above water surface. The caisson is floated and sunk between two barges; to seal it to the part under water the top of the under water part is covered with an asphalt composition into which the caisson edges can sink.

Sheetpile Studies—Driven as described the sheeting was subject to hydrostatic head for its full depth. This depth plus 3⅓ ft. was assumed and the piling was designed to resist it at stresses near the yield point of the steel. The holding resistance of the clay in which the pile was footed could not be ascertained with such accuracy and a factor of safety of 1½ was introduced in the shear values of the clay as determined by the cone test described later. Static calculations based on these assumptions were made to determine the depth to which to drive the sheetpiling.

As a check on these calculations, ob-

FIG. 3—CONE PENETROMETER used to measure the consistency of clay samples from the foundations of the Storstrom bridge.

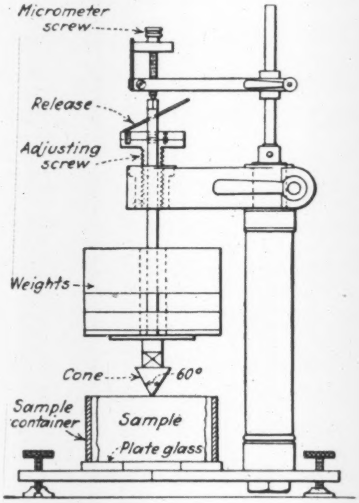


FIG. 4—SECTION OF DAM NO. 3 on the Svir River showing arrangement for reducing uplift and increasing resistance to sliding.

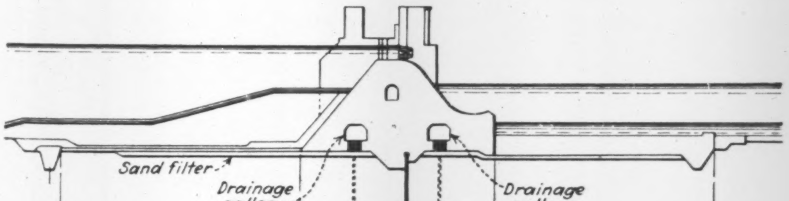
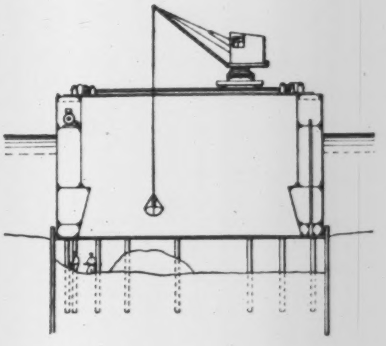


FIG. 5—ANNULAR STEEL COFFERDAM for the foundations of Storstrom bridge in Denmark; coffer sunk by water ballast is unballasted and floated to a succeeding pier operation.



servations of the curvature of many piles were made during excavation and the mean of the readings was assumed to express the stresses produced by water pressure. The piles were generally of soft steel but when very long were of alloy steel having a yield point of about 50,000 lb. per sq.in. The curvature is checked by measuring the midordinate of an arc whose chord is about 5 ft. A tabulation of test results shows in general less stress than the theoretical calculations but it is pointed out that conditions were not identical.

Hydraulic Sampler—To determine the mechanics of the foundation bed clay, over a hundred auger borings were made in which resistance and other data were recorded and during which undisturbed samples were taken. The sampler is designed to take samples about 1.9 in. (48 mm) in diameter. It consists of a steel cutting tube lined with a thin brass tube in which the sample is introduced during the operations. This cutting tube is connected to a piston moving in a cylinder of about 3 in. (76 mm) inside diameter. In the center of the cylinder is a fixed guiding rod ending in a bottom plug, which acts as a piston in the brass tube, when this tube, together with the steel cutting tube and the main piston, is moved downwards in the cylinder by means of water pressure acting on the upper side of the main piston. The water under the main piston will escape through the bore in the guiding rod.

To operate the sampler it is connected by pressure pipes screwed tight together by means of conical threads, and lowered in the casing tube as shown in Fig. 2, with piston locked in its top position and the lower end of the cutting tube flush with the bottom plug, until this reaches the bottom. Soft material left on the bottom is thus displaced. Then a clamping piece is bolted onto the pressure pipe and connected to the loading beams bolted to the casing tube. These beams are loaded by sand boxes or pig iron supplying the reaction to the pressure needed for cutting out of the sample. Now water from the pressure pump is led through the pipe into the cylinder and causes the piston to move so that the cutting tube will penetrate into the ground and cut out a sample of clay. When the piston reaches its bottom position, which is observed on the manometer of the pump by a sudden increase in the pressure, the piston is automatically locked in its position and the sampler is lifted to the deck of the barge, the screw cap is removed and the tube containing the samples detached.

The consistency of the sample is determined immediately on the site by means of a simple apparatus called a spring-scale-cone designed by O. Godskesen (*ENR*, July 23, p. 331), whereafter the brass-tube is sealed with caps and adhesive tape at both ends and sent to the laboratory for further testing.

The clay sampler is designed for a normal working pressure of 50 atmospheres corresponding to a total pressure on the cutting tube of about 2 tons. It was found that the samples taken by this device were considerably less disturbed than samples taken by instruments of more simple design, intended to be driven into the ground by blows with a hammer.

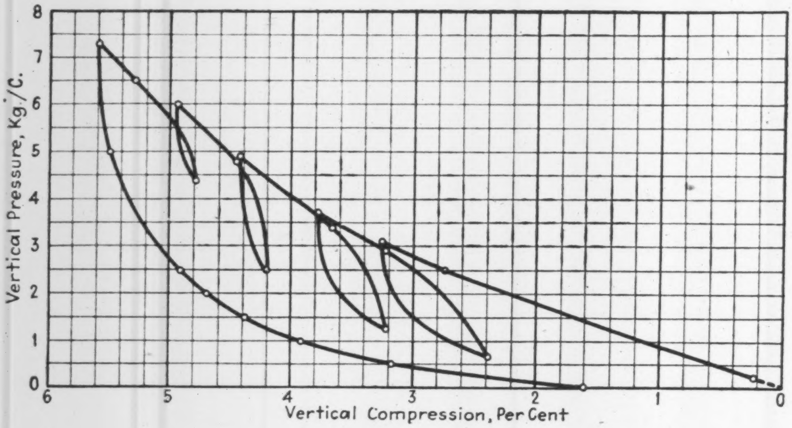


FIG. 6—TYPICAL COMPRESSION CURVE obtained with Oedometer apparatus on representative clay sample from Svir dam foundations.

Cone Penetrometer—In the laboratory the clay sample was put through the usual tests for water content, plasticity, shrinkage, specific gravity, granulometric composition, compressibility and permeability but the main tests of consistency were made with a cone apparatus designed in 1930 by Mr. Bretting (Fig. 3). The tests are made with different weights on the cone and the consistency is defined as the weight of the cone which gives a penetration of 10 mm (4/10 in.) Based on experience and comparative tests Mr. Bretting concludes: "The cone test employed on impermeable clay will express the cohesion of the clay. On account of the great speed of the test the effect of internal friction can be neglected so that results agreeing with the shearing strength observed in actual practice will be obtained. The ratio between cohesion and consistency can however not be considered constant, but must be determined for each type of clay.

Elaborate soil studies made for the dam of hydro-electric development No. 3 on the Svir River in Russia were described in papers by A. F. Samisoe of Sweden and by H. Graftio of Leningra Russia. Much of the important experimentation and discussion does not submit to condensation and here only the foundation structure as it was designed to resist upward water pressure, sliding and settlement is considered. At station No. 3 the river is closed by a concrete dam (Fig. 4) 630 ft. long with a powerhouse at one end and a lock at the other end. The foundation material consists of horizontal layers of alternating clay, sand and clay-sand. In total thickness the clay is about 75 per cent but the layers of the several materials range in depth from a fraction of an inch to a yard.

In character the soil layers are quite different. Generally the thick sand layers are hard, some of them transformed into sandstone. Also certain layers of clay containing iron oxide are consolidated to a great extent and are nearly as hard as shale. Most of the layers are much less consolidated, some of them are compact and plastic, a few of them are soft. Swelling was a distinct characteristic of the clay when relieved of load, and consequently contraction when reloaded. Again the sand layers contain artesian waters. In brief, the foundation conditions indicated ample strength under vertical load but the danger of sliding from the horizontal slick layers, upward water pressure and swelling and settlement. Elaborate soil studies were made to determine the possible extent of the hazards, particularly in respect to swelling and settlement.

As shown by Fig. 4 the conditions were met in design by giving the dam a base as flat as possible. To increase the resistance to sliding, abutting upstream and downstream soil and water loaded concrete aprons were provided. To provide against uplift a bed of sand placed under virtually the whole area of dam and aprons. This sand layer is drained to two drainage galleries in the dam and these galleries empty downstream; incidentally the sand layer serves as a filter to prevent loss of fines from the ground below. As a further release of underground water a row of sand wells was constructed under each drainage gallery.

In construction the water was kept from the foundation pit by a well-point drainage system. Around the pit 12-in. wells were sunk to 40 ft. below foundation level, well points were inserted and the holes were filled with sand.

Discussing the vitally important subject of settlement Professor Graftio says:

Great difficulties were due to the clay being apt to absorb water and swell when relieved of load, and to discharge water and contract when reloaded. In the first stages of design it was evident that considerable settlements of the structures must be expected, and therefore an extensive series of tests was made to determine the elastic properties of the ground.

The results of the tests may be summarized as follows: When, due to varying loads, clay or clayey layers absorb or discharge water, a change in the volume of voids is effected. The corresponding upheaval or settlement of the ground is nearly fully elastic and is characterized by a great hysteresis. The aptitude of the soil to contract and swell varies and is dependent upon the consolidation of the layers and upon their sand content. A diagram typical for most of the clayey layers is shown in Fig. 6. To give an idea of the compressibility of the ground it may be mentioned that settlements measured so far have reached: Under the power house 10 in., under the dam 9 in., under the lock 11 in.

It is obvious that the erection of a power plant on an elastic ground in which the settlement in some places attains 11 in. may cause anxiety in regard to the possible occurrence of cracks in the concrete structure, the opening of joints and so forth. Therefore it was necessary to attempt to find out beforehand how the settlement under the various parts of the concrete structure would proceed during the different stages of the construction work, and later on, in the course of impounding the water upstream of the plant. The calculations relating hereto were chiefly made in the following way.

In a number of places along certain cross sections the change of the vertical pressure in the ground was computed on the assumption that the ground under the concrete structures consisted of an elastic homogeneous material of unlimited extent. The settlement of the surface of the ground was calculated on the basis of Oedometer tests, and according to a method devised by A. Frey Samsioe.

There was never a doubt but that the result of these calculations must be uncertain. In the first instance, because the ground is not homogeneous, in the second because the subterranean water pressure during different stages of construction influences the ground in a manner that cannot be adequately dealt with in mathematical formulas. In order to ensure every obtainable means of adjusting the method of calculation to existing conditions, it was deemed necessary to observe very closely the movements of the ground during the early stages of the work. At different depths in the ground gages were arranged and leveled at close intervals, as the excavation and concreting proceeded, and later on auxiliary gages were applied to various concrete structures.

Through these continuous investigations it was established that the actual settlements did not reach the limits originally computed. In the sense of relative values, however, the agreement was very close. That the computed settlements proved too great may be due partly to the unequal consolidation of the different layers, partly to the fact that the ground was loaded with concrete immediately after the excavation had been completed, and thereby dispossessed of the possibility to swell to its ultimate volume. However, it was very useful to be able to establish, at an early stage, that everywhere 35 per cent of the computed settlements corresponded very nearly to the actual settlements.

Thus means were provided for planning and organizing the work in a manner promising the least possible deformation of the structures. In the power house and the dam, the total length of which is about 1,055 ft., there is only one expansion and contraction joint.

Generally the concreting was carried out in monoliths, the lower parts in smaller units, separated from each other by open joints which, when suitable in regard to settlements, were concreted.

For certain reasons it became necessary to construct the power house and the concrete dam in advance of the aprons to which they were to be anchored. Therefore, and because the aprons would be subjected to an increasing load until the water above them reached its ultimate level, very uneven settlements were to be expected in the transition zones between the aprons and the other structures. Obviously it was of paramount importance to ensure here a most elastic water-tight connection.

This was effected by making the chief constituent of the connection of asphaltic concrete on top of which a layer of pure asphalt was laid. As the combined load of earth fill and water subjects the asphalt to a considerably higher pressure than the water pressure corresponding to the difference between the upstream and downstream water levels this very flexible connection may be deemed absolutely watertight as long as the asphalt remains, which should be forever, the asphalt being well protected against exterior agents of injury.

The connection between the aprons and the concrete cutoff walls upstream of the aprons are designed on the same principle; also the expansion and contraction joints between different parts of the aprons. It may be noted that in every individual case great care has been taken to prevent the asphalt from flowing sideways if, for one reason or another, the vertical load on the asphalt sheet should change from place to place.

The aprons and the lower part of the upstream surfaces of the power house and the dam were covered by asphalt mats, and on top of these layers of puddled sandy clay were arranged. According to observations made, there is every reason to believe that the watertightness in these parts is perfect.

Numerical Value of Web Splices

By Frederick L. Hough
Brooklyn, N. Y.

IN INVESTIGATING the strength of existing structures the problem often arises as to what value to allow for web splices on plate girders. In using the flange area and effective-depth method of calculation, it is customary to add one-eighth the web area to the area of flange angles and coverplates. But the web splice may not be equivalent to one-eighth of the web area. What equivalent area then should we use? Or, if we are calculating the moment of inertia of the girder, what equivalent moment of inertia should be used for the splice rivets?

In determining a value for A_s, the equivalent flange area of the splice rivets, the value of one rivet a distance x from neutral axis is $(R/d)\ x$, where R is the maximum rivet value and d is the distance of the flange from the neutral axis. The resisting moment of one rivet is $(R/d)\ x^2$, and the total resisting moment of all rivets $(R/D)\Sigma x^2$. The equivalent resisting moment for the girder is A_sSD, where S is the allowable flange stress and D the effective depth.

$$A_sSD = (R/d)\Sigma x^2$$

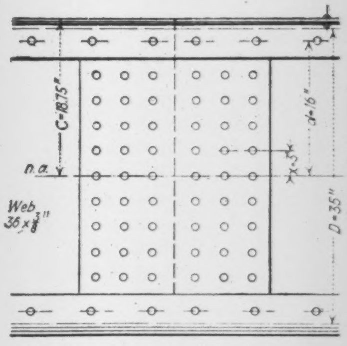
and
$$A_s = \frac{R}{SDd}\Sigma x^2$$

To find the moment of inertia, I_s, of the splice rivets in the same units as for the plates and angles, the equivalent moment SI/C is used.

$$SI/C = (R/d)\Sigma x^2$$

$$I_s = \frac{Rc}{Sd}\Sigma x^2$$

As an illustration of the use of the formulas, take the girder shown in the sketch. Rivets $\frac{3}{4}$ in. in diameter are used which have a maximum value at 24,000 lb. per sq.in. bearing of 6,750. $\Sigma x^2 = 6\ (3^2 + 6^2 + 9^2 + 12^2) = 1{,}620$.



TYPICAL girder web splice whose equivalent flange area is less than one-eighth of the web.

Then

$$A_s = \frac{6750 \times 1620}{18000 \times .35 \times 16}$$

$$= 1.09 \text{ sq.in. } (1/8 \text{ of web} = 1.69)$$

$$I_s = \frac{6750 \times 18.75 \times 1620}{1800 \times 16}$$

$$= 708\ (I \text{ of web} = 1458)$$

At first glance the splice shown appears quite adequate to develop the web, and yet the investigation by either method shows it weaker than the web.

The above values used for R in the calculations are at a point where there is no shear. If shear is present, the total vertical shear at the point of splice must be divided by the total number of rivets on one side of the joint to obtain the shear per rivet; the allowable value for R is then obtained by the usual method of resolution of forces.

FORT PECK DAM TODAY—IV

Grouting the Tunnels

By J. D. Jacobs

U. S. Engineer Dept.,
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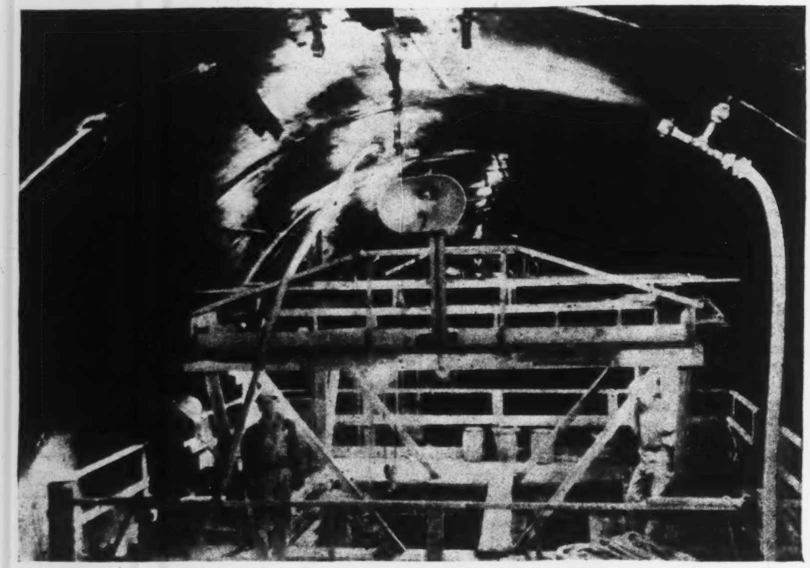


FIG. 1—GROUTING the Fort Peck diversion tunnels is done from a traveling jumbo that clears concrete and muck trains below.

EVEN BEFORE the concrete lining for the Fort Peck diversion tunnels was started it became apparent that the subsequent job of grouting the arch voids would be a task of unusual proportions. The complicated steel excavation bracing, with its supplementary timber and blocking, and occasional overbreaks, has made it impossible to place the arch concrete without developing a continuous cavity between the top of the concrete and the shale. Investigations have shown that the volume of the void averages about ¾ cu.yd. per lin.ft. of tunnel; if not eliminated this void would form a very undesirable water passage outside the lining. The application of the 500,000 cu.ft. of grout necessary to fill these openings has required the ingenious solution of many problems.

Confronting the engineers who designed the original grouting plant was the problem of selecting equipment that would economically manufacture and place the grout in each of the four tunnels rapidly enough to keep pace with the forerunning operations of excavating and concreting. The four tunnels are 24 ft. 8 in. in diameter inside the lining, and their lengths range from 5,300 ft. to 7,200 ft. Whatever grouting machinery was to be placed within the tunnels had to be portable to move forward with the progress of the work. Moreover it was necessary that the traveler carrying the equipment should not interfere with the traffic over the double-track narrow-gage railroad which serves the operations ahead. The question of whether to build a single rig to be moved from tunnel to tunnel or to provide similar and individual plants in each tunnel was decided in favor of the latter method after an investigation of the cost of shifting equipment and the time lost in doing so.

It was decided to place all necessary handling, batching and mixing machinery at the point of application of the grout inside the tunnel, all the equipment of a unit to be mounted upon a traveling steel-frame jumbo. A central sand storage bin was built outside the tunnel portals and a gas-fired drier was installed above the bin to enable the sand to be handled during freezing weather and also to destroy any vegetable matter such as weed roots, etc., which might have accumulated therein. Although the first-tried method, with individual inside batchers, was discarded after a brief trial, their design is interesting enough to warrant a description.

To give the necessary 13 ft. horizontal and 9 ft. vertical clearance over the service tracks, the jumbos were designed as a steel gantry frame with flanged wheels riding the 14-ft. gage form rails which had been left in place for their accommodation. Two grout mixers were mounted on each jumbo, at a height of 6 ft. above the track and 17 ft. apart, on opposite sides of the tunnel to clear the trackway. At an elevation of 10 ft. above the track, a working deck extended across the tunnel and upon this deck were mounted the sand and cement weigh batchers and a short transverse belt conveyor, powered by a reversible air motor, for distributing the batch to the mixers. In the top of the tunnel above the batchers were placed steel bins for temporary storage of sand and cement.

A novelty in this arrangement, and also the feature which caused the abandonment of the individual batching in favor of the central mixing and batching plant now in use, was a pneumatic conveying system for transferring the sand and cement from cars to the bins in the top of the tunnel. The top of the sand storage bin was 22 ft. above the track level, the top of the cement bins 18 ft. For elevating the materials this height, various devices were considered including bucket elevators, skip hoists, screw conveyors and pneumatic conveyors. Because of cramped working quarters and no power other than the 110-v. lighting circuit, the pneumatic method was the one chosen.

It was proposed to haul the bulk cement and sand into the tunnels in tank cars equipped with outlets at the bottoms for the conveyor pipe which would carry the materials to the overhead bins. The bins were capped and vented through air filters to reduce the dust nuisance. Early experiments had indicated this method to be workable, but when it was attempted to put it into practical use one of the sand tanks exploded due to back pressure of the air. After this mishap the system was immediately abandoned for a safer one.

The decision to adopt a central batching and mixing plant outside the tunnels meant the reconstruction of the inside placing jumbos. The two mixing machines were left on the jumbo to be used as placers, but the belt conveyor, batchers and bins were removed. Two trans-

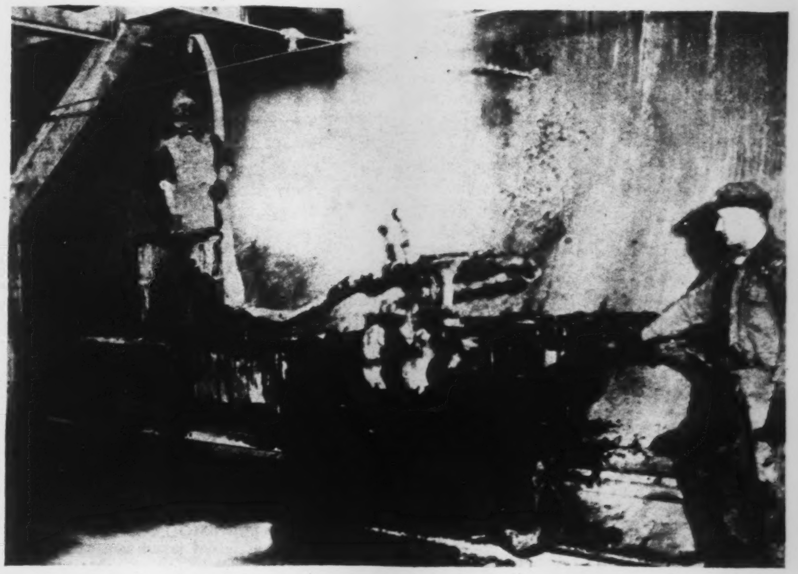


FIG. 3—HIGH-PRESSURE grouting is done by air-operated plunger pumps mounted on three-wheel carriages which ride the gantry rail and still clear muck trains.

verse trolley beams were installed across the top members of the frame, from which were hung two 3-ton electric hoists for lifting the buckets of ready-mixed grout from the flat cars below to the charging hoppers above the mixers. A second working deck was built on top of the jumbo frame above the hoist beam, accessible to the grout connections in the roof of the tunnel.

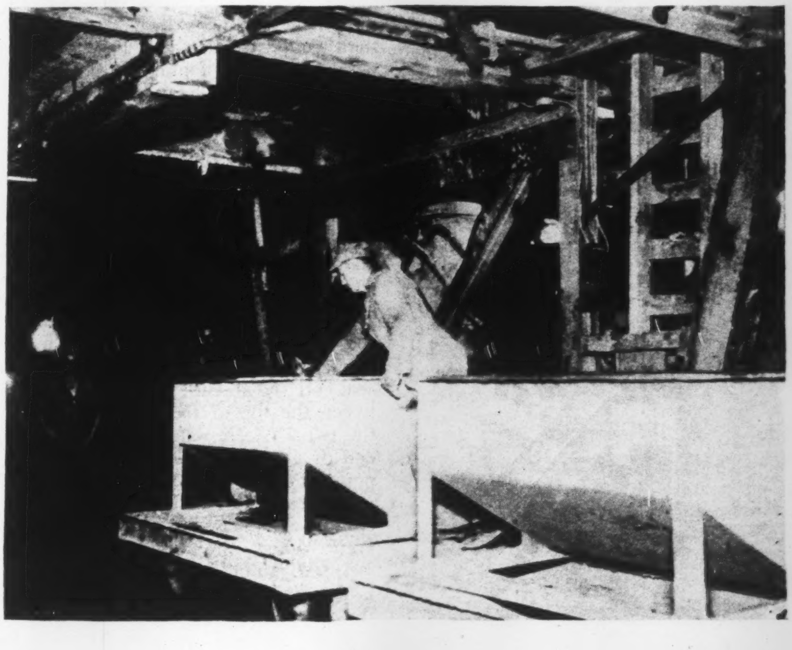
In addition to the steel grout-placing jumbo, two more auxiliary platform jumbos are necessary for access to the grout holes both in front of and behind the placer. These are of wood construction, mounted on wheels, have the same track clearance as the placing jumbo, and are light enough to be pushed manually by the workmen. The heavier placing jumbo is moved by means of an air winch mounted on the jumbo frame.

The central grout-mixing plant is located alongside the original sand bin and drier structure. Similar to a concrete batching and mixing plant, with the omission of gravel batching equipment, the building is of timber construction, completely housed in for cold-weather operation. Equipment includes a 43-cu.yd. sand bin and a 320-bbl. cement bin in the top of the structure, feeding directly into two bin-type weigh batchers handling 1,000 lb. of cement and 2,000 lb. of sand. Beneath the batching and operating floor is a standard $\frac{3}{4}$-yd. concrete mixer powered by a 15-hp. electric motor. The mixer dumps into an outside steel hopper above the tunnel service track and the grout discharges into the buckets on the flat cars through a gate valve in the bottom of the hopper. All the controls for the mixer and gates are located on the batching floor; one man regulates all operations of the plant.

Sand is transported to the plant by dump truck from a stock pile 300 ft. away. Loading at the pile is by means of a self-propelled bucket elevator loader. Cement is received in bulk in carload lots and is conveyed pneumatically through a 5-in. pipe line 800 ft. from the unloading dock to the mixing plant bin.

Transporting the mixed grout from the central plant to the placing jumbos in the tunnel is done in open buckets on standard 6-ton flat cars, one bucket to a car and four cars to a train, pulled by combined trolley and battery mine locomotives. The buckets, designed and built on the job, are welded up of $\frac{3}{16}$-in. steel plates and frames of light angles, $4\frac{1}{2}$ ft. square and 3 ft. high. This unusually shallow design was necessary in order to conserve headroom for hoisting inside the tunnel. In spite of their shape, however, the buckets have been found to handle satisfactorily without excessive slopping. They have a V bottom and discharge through a 4-in. iron steam cock at one end. Each bucket has a capacity of 21 cu.ft. of grout, or one mixer batch.

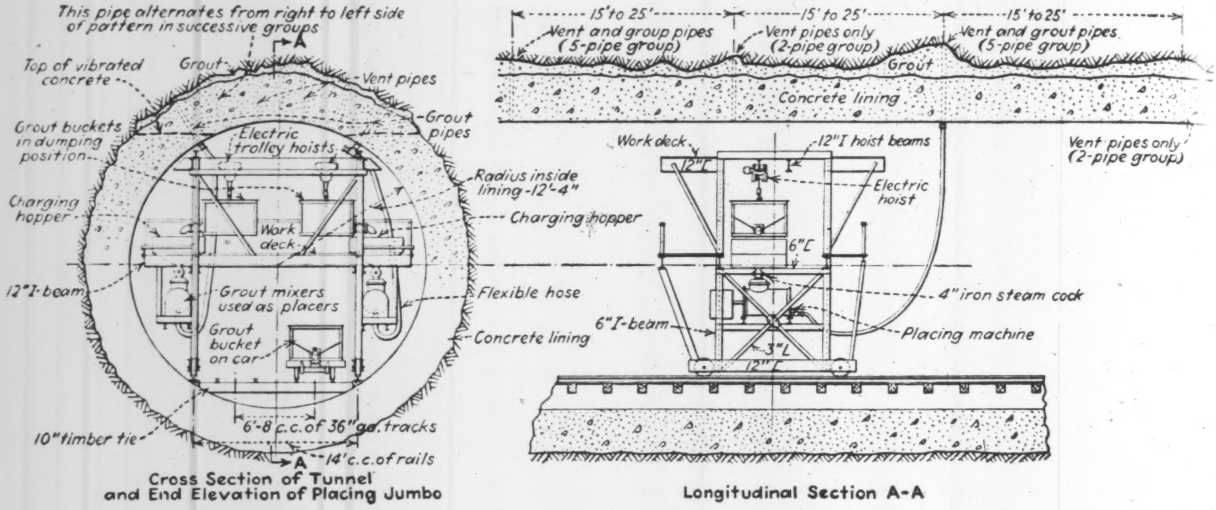
FIG. 2—PLANT-MIXED grout is brought to the jumbo in shallow buckets, which are hoisted from their carrying cars for dumping into twin grouting machines mounted on each side of the jumbo.



A small amount of settlement takes place in the buckets during transportation, but this is not detrimental to the grout. At the placing jumbo the buckets are drained into receiving hoppers above the mixers. Air jets have been installed in the bottoms of the hoppers and the boiling action of the air is an effective means of re-mixing the materials before they are admitted into the air chambers.

Preparation for grouting

Due mostly to the shrinkage of the grout in setting, it has been necessary to follow the primary or low-pressure operation with a secondary high-pressure application of neat grout. Primary grouting, which comprises more than 95 per cent of the entire volume placed, is done with the equipment on the placing jumbos described above. Pumps for the high pressure grouting will be described later. The primary or low-pressure application uses a sand grout, proportioned with equal volumes of sand and cement and 65 gal. of water to a 21-cu.ft. batch. Neat grout for the high-pressure work contains 12 to 20 gal. of water to the sack of cement.


This pipe alternates from right to left side of pattern in successive groups
A
Top of vibrated concrete
Grout
Vent pipes
Grout buckets in dumping position
Electric trolley hoists
Grout pipes
Radius inside lining -12'-4"
Charging hopper
Charging hopper
Work deck
12" I-beam
Grout mixers used as placers
Flexible hose
Grout bucket on car
Concrete lining
6'-8 c.c. of 36" ga. tracks
10" timber tie
A
14' c.c. of rails
15' to 25'
15' to 25'
15' to 25'
Vent and group pipes (5-pipe group)
Vent pipes only (2-pipe group)
Vent and grout pipes (5-pipe group)
Grout
Concrete lining
Work deck
12"I hoist beams
12"[
Electric hoist
Vent pipes only (2-pipe group)
6"[
4" iron steam cock
6" I-beam
3"L
12"[
Placing machine

Cross Section of Tunnel and End Elevation of Placing Jumbo

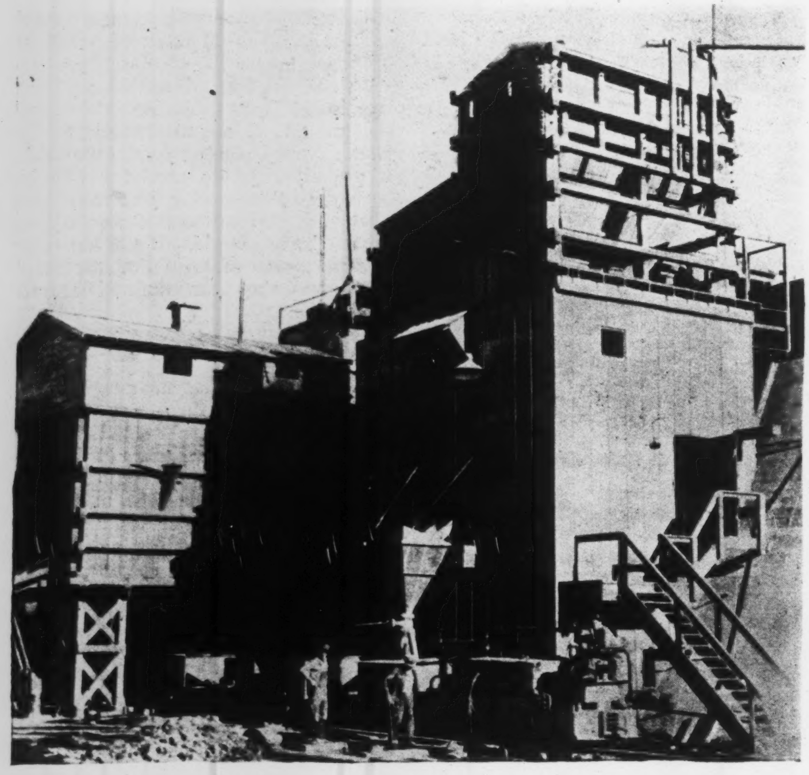
Longitudinal Section A-A

FIG. 4—JUMBO-MOUNTED grouting rig uses grout mixed at an outside central plant. This sketch shows the various features of the rig and details the grouting procedure.

At the start of the operations the grout holes were drilled through the concrete lining from a portable drilling jumbo. Attached to the frame of this machine were adjustable bars to which were fastened jackhammer drills mounted on screw-feed shells and equipped with hollow-centered $2\frac{5}{8}$-in. four-point bits. After the holes had been drilled, 2-in. pipes were inserted for the hose connections. The recent change in the design of the tunnel linings, which resulted in placing a 45-in. lining instead of the original 21-in. shell, gave more working space above the form, enabling the 2-in. grout pipes to be placed before the pour and cast into the arch. This procedure has eliminated the necessity of drilling the holes through the concrete.

The pipes are placed in groups, the number per group alternating two and five, and the spacing of the groups varying between 15 ft. and 25 ft. All the pipes in a group are placed in line circumferentially in the top of the arch. The two-hole groups act as vents and their pipes extend to shale. The two vent pipes are usually vertical and spaced about 4 ft. apart, symmetrical with the center line of the tunnel. In the groups of five, the two uppermost pipes act as vents, the other three are for placing the grout. The two outer pipes are located symmetrically with the center line and down about 35 deg. from the crown of the arch. The fifth or odd hole, which is also a grout pipe, occurs between the vents and the outer grout pipes and its location alternates from the left to the right side of the pattern in successive groups of five (Fig. 4). At intervals of 50 ft. along the axis of the tunnel, one of the two vent holes is drilled 10 ft. into the shale as a test hole to detect possible fissures resulting from settlement of the crown rock. When such voids are detected they are filled with neat grout applied with the high-pressure pump. Detailed geological and cross-sectional surveys of the tunnel excavation, which have been plotted in the office, are the basis for determining the location of the hole groups, and give valuable assistance in planning the field operations.

FIG. 5—CENTRAL PLANT for mixing the grout, located outside the lower portals. The sand drying house is at the left.



Low-pressure operations

To confine the flow of grout to the limits of a single pour, dams of mortar are constructed at intervals to seal off whatever opening exists between the top of the arch concrete and the rock. These dams are placed at the end of a 70-ft. section of concrete arch pour and are made by hand packing and troweling fresh mortar into the cavity which remains above the concrete when the bulkhead has been stripped. The location of these dams is determined in advance by a study of the overbreak conditions shown in the profiles. Each grout pour is planned to be of such length that it can be completed within six shifts, or 48 hr. of continuous pouring. This usually is obtained by placing the dams 280 ft. to 420 ft. apart.

The grouting operations, started at the outlets of the tunnels, are proceeding upstream following the concreting operations. At the start of a pour, the grout placing jumbo is moved beneath the first set of grout holes upstream from a dam and two hose connections are made, one from each machine to the lower grout hole on its side of the tunnel. Grout is then applied until each hole has refused at the application of 50 lb. pressure. The machine is next moved forward to the first grout pipe upstream which has not vented, at which point connections are again made and grouting is continued until all high pipes downstream have vented. In case any of the downstream high pipes fail to vent, connection is made to them and continued until refusal. The vent holes, which serve the dual purpose of releasing trapped air and indicating the position and extent of the grout flow, are sealed with wooden plugs as soon as venting begins.

The grouting operation is continued step by step upstream until the next grout dam has been reached, at which time the low-pressure crew leaves the completion of that section to the high-pressure gang, while they shift their efforts to another tunnel. In the low-pressure work, pressures in excess of 50 lb. per sq.in. are never used.

High-pressure grouting

By waiting no longer than is necessary for the fresh grout to set into a putty-like solid (usually 8 to 10 hr.) the redrilling of the pipes preparatory to the high pressure application is made an easy matter. For this purpose a jackhammer drill is used, equipped with a twisted steel auger bit. Access to the holes is by way of the aft jumbo described above.

The device for placing the high-pressure grout is a 3x6x3-in. reciprocating plunger pump actuated by compressed air. Since only a very small volume of grout is applied by this method, all mixing is done inside the tunnel at the point of application. Mixing is manual in a round vertical tank improvised from half an oil drum and equipped with a crank-operated vertical paddle shaft in the center. The freshly-mixed grout is drained from the mixing tank into a square sump tank between the mixer and the pump, into which the suction hose from the pump extends. The suction hose is fitted with a 50-mesh wire screen strainer to remove foreign particles. Agitation is accomplished by admitting compressed air into the bottom of sump tank. The high-pressure pump and tanks are compactly arranged on a carriage equipped with two wheels which ride the form rail on one side of the tunnel, and a third balancing wheel which runs along the concrete wall of the tunnel. The whole arrangement sits back out of the way of passing trains.

Two passes are made over each section with the high-pressure pump. The first pass starts at the upstream end of a section and proceeds to the downstream end, connections being made to all pipes and pumping continued until refusal. The second pass begins at the downstream end of the section and returns to the upstream end, but connection is made only to those holes which are in or adjacent to fault areas in the shale. The pressure of application is determined by the condition of the ground and the amount of cover above the tunnel. The usual maximum is 75 lb. per sq.in. For high-pressure grouting in fault zones, a thinner mix of water and cement is used than in ordinary ground.

Whenever a fault plane crosses a tunnel the fault zone is sealed with neat grout through a set of five radial holes drilled 10 ft. into the shale. These holes are located to contact the fault zone 6 ft. from the inside of the tunnel lining. One hole is drilled downward through the invert, one horizontally in each side of the tunnel at the center line, and two vertically in the arch, the higher of the latter two acting as a vent hole. These fault holes are grouted with the high-pressure pump.

The cement used is the same in composition as that used in all concrete in the Fort Peck job, manufactured to specifications designed for alkali resistance. The sand is a river deposit of exceptionally fine rounded particles and is obtained from beds near the site. All sand has been screened to remove foreign objects and vegetable matter. This sand produces a grout capable of being forced long distances without segregation, making it more satisfactory than a sharp coarse sand, the grains of which have a tendency to sink to the bottom of the moving mass of grout while the cement and water run on ahead.

Organization

Like all construction operations on the Fort Peck tunnels, the grouting is being done with hired labor by the U. S. Army Engineers. Capt. A. W. Pence is in direct charge of tunnel construction under Lt.-Col. T. B. Larkin, district engineer for the Fort Peck project. George J. F. Carey is engineer of concrete and grouting work, and R. E. Tual handles the grouting operations in the field.

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Steel-Frame House Costs

By C. Merrill Barber
Structural Engineer, Cleveland, Ohio

IT HAS BEEN the privilege of the writer within the last year and a half to make the structural design for two large steel-frame houses in Ottawa Hills, a suburb of Toledo, Ohio. Myron T. Hill was the architect for both houses. The first house contains 53,000 cu.ft. and costs about $26,000. Fifteen tons of steel were used in the floors and walls of the frame. The additional cost of using steel frame over the cost of a wood frame was 3½ per cent of the total cost of the house.

The second house, containing 140,000 cu.ft., cost about $90,000. Many of the important rooms in this house have clear spans of 19 ft., necessitating the use of heavier floor and wall steel than is usual in residential design. This resulted in a reduction of economy in terms of prices per square foot in comparison with the first house. Forty-five tons of steel were used, of which 62 per cent is in the floors, 33 per cent in the walls, and 5 per cent in loose lintels and clips. Floors are constructed of 8-in. channels or beams spaced 3 to 3½ ft. on centers, while the walls are made up of large shop-welded panels one story high using 3-in. channels as studs and angles, plates, etc., as necessary. The largest panels weighed only 500 lb. and were about 10x18 ft. The total erection required 10 working days for four structural iron workers and a foreman. Field welding, of necessity an intermittent process, required one welder for ten days.

The accompanying tabulation shows the breakdown of the steel costs, which total $5,600. It was estimated by the contractor that a comparable wood frame would cost $3,600, so that the premium paid for steel-frame and fireproof construction was but $2,000, or 2.2 per cent of the total cost of the house.

STEEL COSTS FOR A $90,000 RESIDENCE

Erection	45.1 tons @ $21.95/ton..	$990.00
Shop paint	45.1 tons @ 4.66/ton..	210.00
Field paint	45.1 tons @ 4.66/ton..	210.00
Material Costs and Shop Work		
	Lb.	
First floor	15,400	
Second floor	20,800	
Third floor	19,400	
Lintels	4,100	
	59,700 lb. @ $56 a ton..	1,671.60
Exterior walls	19,213	
Interior walls	10,787	
Clips	500	
	30,500 lb. @ $148.74 a ton	2,268.40
Field welding	45.1 tons @ $5.54 a ton..	250.00
Grand Total,	45.1 tons @ $124.00 a ton	$5,600.00

CURRENT NOTES ON SEWAGE DISPOSAL

An analytical summary of recent developments conducted by

WILLEM RUDOLFS
Chief, N. J. Sewage Experiment Station
New Brunswick, N. J.

● ODOR CONTROL BY DILUTION

When control of disagreeable odors around sewage plants is considered, the addition of chlorine or other chemicals comes to mind immediately. That other equally effective and less costly methods may be employed is shown by Sperry (*Fifth Annual Report Aurora, Ill., Sewage Treatment Plant, 1935*). To avoid particularly offensive odors, an additional 1 to 1.5 million gallons of river water is pumped into the plant to dilute the 6 mgd. of sewage. A careful analysis of pumping cost during full plant operation for 27 months shows that the additional cost for pumping the river water amounted to about $50 per month. Experience has shown that this method of operation has been successful, because the city won the $20,000 odor suit brought by nearby proprietors of resort cottages (*ENR*, Jan. 9, 1936, p. 68). Approximate calculations indicate that the cost of odor control by chlorine would be several times the cost of pumping sewage.

● PRETREATMENT OF ABNORMAL SEWAGE

During a normal dry-weather period it oftens happens on one or two days per week that both the volume and strength of sewage arriving at the treatment plant are greater than for the remaining days of the week. With sewage originating in a residential area such increases in volume and strength may be due to "wash days"; in other cases they may be caused by periodic industrial activities. To produce at all times a satisfactory effluent consequently may involve the installation of greater plant capacity than is normally required. Ardern and Lockett (*The Surveyor*, April 3, 1936) found that pretreatment of sewage handled in an activated sludge plant could be accomplished economically by (1) adding a coagulating agent to the sewage, (2) aerating the mixture of chemical and sewage for a very brief period, and (3) terminating with a short period of settlement.

Of the various coagulants tried it was found that acid with aluminoferric gave best results. With the sewage thus treated best clarification was obtained at pH 6.4: the alkalinity of the sewage was reduced by half with sulfuric acid and thereafter with the coagulant. Gentle stirring or aeration for 3 to 10 min. after the addition of the chemical caused more rapid clarification with saving of chemical. The sewage after being settled for about 30 min., in no way impaired the purifying properties of the activated sludge. In view of the temporary nature of the pre-treatment —from three to six hours for one or two days a week—this is not thought to be of serious consequence in connection with either bulk or low alkalinity of the sludge resulting from the chemical treatment. Partial part-time treatment of this type should be more economical than making provision for greater plant capacity to deal with occasional overloads.

● HARBOR SICKNESS AND POLLUTION

Harbor sickness occurred in Germany for the first time during 1924, gradually disappeared and then an epidemic occurred in 1932, chiefly among fishermen around fresh water harbors. To date more than 1,000 cases have been reported with 13 deaths. More than 80 per cent of the sick persons were men, the remainder women. Strangely enough no children were affected. During the second outbreak thousands of gas masks were distributed in the belief that the sickness was presumably caused by arsenical gases produced from industrial wastes discharged into the harbors. However, Emde in a paper before the Water, Sewage and Sanitation Section of the Germany Chemical Society (*Vom Wasser*, vol. 10 p. 37-44) states that the same sickness occurred in Konigsberg, where the harbor does not receive trade wastes. Studies seemed to indicate that fir wood used in the cellulose factories contains certain resinous acids, which causes the sickness.

From a chemical standpoint objections have been raised that these resinous substances could be the cause of the illness. Experiments with arsenic gas-air mixtures on cold and warm blooded animals, did not show any similar effects, whereas eels and water fowl feeding in polluted water became sick and died in large numbers. In addition several human cases occurred with only remote relation to fresh water harbors. Eels kept in polluted water died, and cats fed with these eels became sick.

Not all fresh harbors receiving polluting matter caused the sickness, and it is believed that the poisonous substances are developed only where an accumulation of polluting substances takes place. These secondary poisonous substances appear to occur when decomposition of sewage or trade waste sludge is active.

● VACUUM TRICKLING FILTER

An experimental vacuum trickling filter, filled with slag and tightly closed, has been built at Zurich to compare results with an experimental activated sludge and a chemical coagulation plant. According to Husman (*Gesundheits Ingenieur*, May 16, 1936) the most important questions to be answered are: relative cost, permissible load, effect of trade wastes, volume of sludge produced, is introduction of heated air beneficial, and can a higher degree of purification be reached than with ordinary trickling filters.

The settled sewage discharged from the Zurich plant causes excessive growth of fungi in the water which is used in part for drinking purposes. Four different artificial "streams" or channels have been built to determine the cause of the growth and the effect of the different types of treatment.

● MILK WASTE TREATMENT

About ten different methods of milk waste treatment, varying from broad irrigation to the activated sludge process, are in use. In addition several combination treatments are employed. Treatment on trickling filters frequently results in difficulties due to the accumulation of fat in the top layers of the filter. At other times the alkali in the wash waters interferes with the biological action in the filters. Barritt (*Jour. Soc. Chem. Ind.*, Mar. 6, 1936) describing experiments on the treatment of milk factory effluents by means of trickling filters following septic tanks, found that preliminary treatment in a septic tank brings about separation of the fat and thus avoids clogging difficulties. The use of alkali wash waters favors the proteolyses in the storage tank, but the addition of wastes containing sugar will result in abnormally high acidity which may cause the precipitation of the casein. Such tank effluents, especially if high in biochemical oxygen demand have a wide carbon-nitrogen ratio and are not particularly suitable for treatment on a trickling filter without the addition of nitrogen and probably also of phosphates. The use of two filters, operated in series, may overcome the difficulty. When some solid matter has accumulated in the first filter the order of use is reversed, so that the first filter is supplied with treated effluent from the second filter. This causes the solid matter to disintegrate and oxidize and thus the filter becomes cleared.

From Job and Office

WPA Street Marking Project Aids Traffic Movement

By W. B. Hendrick
City Engineer, Lawton, Okla.

Unmarked street intersections in Lawton, Okla., led to considerable confusion to large numbers of people who passed through or visited the city each year. Consequently traffic movement through the city was hindered and in order to overcome the inconvenience which resulted, in part, from expanded city limits and off-set street extensions, a street marking project was submitted to the WPA calling for the identification of 400 intersections. This project, proposing the marking of 365 intersections in the residential area with concrete marker posts on diagonal corners, and 35 intersections in the business district with two-way metal markers attached to electric light posts on diagonal corners was submitted under the sponsorship of several civic organizations.

The concrete markers are triangular, with 8-in. faces, and all exposed edges have a 1-in. chamfer. The posts are 4-ft. in length, and are buried 18 in. in the ground, leaving 30 in. for the vertical street names. The markers, of 1:2:4 mix with three 42-in. reinforcing bars of ½-in. diameter in each post, were cast in single forms. After being removed from the forms the markers were given a carborundum finish and placed in moist sand for curing. After erection the posts were painted with asphaltic base aluminum paint and the names were then stenciled on with black paint. In placing these markers care was taken to set them far enough back from the curb so that surface water and mud would not be splashed on them, and yet near enough to the roadway so that they would not be out of night driving vision.

Metal markers, for use on electric light poles in the business section, were of cast iron and were 5 in. high and 20 in. long. The signs had a ¼-in. raised border, and the letters, which were 3½ in. high, were also raised ¼ in. The markers were painted black, with the letters and the border in aluminum. Two markers were joined at the corners with light angle iron and then fastened to the post by a circular clamp.

The city throughout the course of a year has a large visiting public because of the numerous attractions nearby. Fort Sill, located about 5 mi. north of the city, brings about 8,000 National Guardsmen and C.M.T.C. students to every summer encampment. The U. S. Biological Survey, located in the Wichita Mountains 16 mi. northwest, attracts 25,000 visitors every year, and the Wichita Mountain Sunrise Easter Service also draws a large number of visitors; the attendance to this service this year was 123,000. The greater portion of these visitors pass through the city of Lawton and the need for well marked streets has been great for some time past.

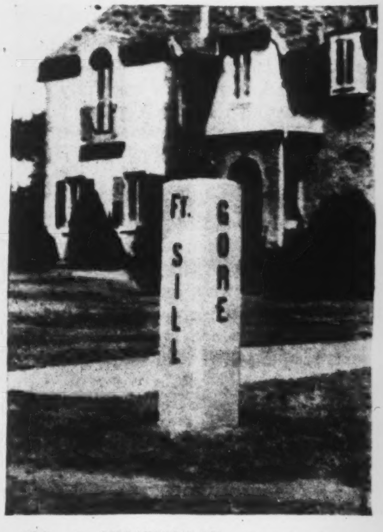


FIG. 1—TRIANGULAR concrete posts, painted in black and aluminum, mark street intersections in the residential sections of Lawton, Okla.

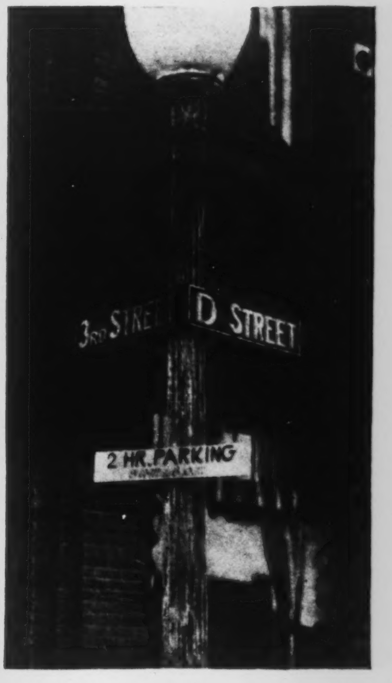


FIG. 2—Cast iron signs, with legible, aluminum painted letters on a black background, are clamped to lamp posts to identify streets in the business section.

◆

Dynamite Bombs Aid in Dredging Small Stream

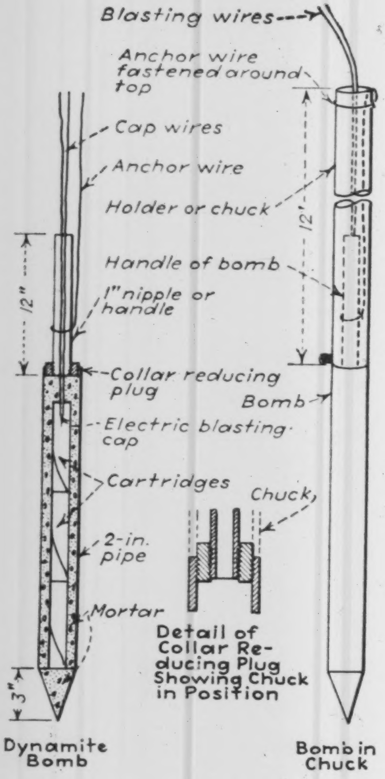
The use of dynamite bombs of 2-in. pipe charged with cartridges proved successful in dredging shallow Wolf Creek for the construction of the Shourt Route Highway in Athens County, Ohio, after other equipment had failed, according to an article by F. R. Cozzens in the March, 1936, issue of *The DuPont Magazine.*

It was necessary to deepen the channel at Wolf Creek at three points for a total distance of 1.3 miles. The stream is winding and rocky, has an average width of 50 ft., and varies in depth from 3 to 8 ft. The channel bed is of shale and sandstone, too rocky for efficient dredging and too irregular for ordinary ditching.

Specifications called for a uniform channel 8 ft. in depth, and after three different contractors had abandoned the project because of inefficient equipment, the Johnson Co., of Trimble, Ohio, accomplished the task in record time by the use of dynamite bombs. The actual excavating was done in 140 working hours, using 1,700 lb. of nitroglycerin dynamite, 600 electric blasting caps, 6,000 ft. of copper wire and 1,500 ft. of 2-in. iron pipe.

Bombs made on job

The bombs were made in advance, using 30-in. lengths of 2-in. pipe, the bottom ends of which were welded to a point after cutting out four gores. Next, cement mortar was poured to a depth of 3 in. and permitted to harden. Then, in each shell was loaded, end to end, the required number of cartridges, and the last one inserted carried an electric blasting cap with insulated copper connecting wires. To seal the charge, more mortar was poured in, and then a 1-in. reducing plug was



DETAILS of dynamite bomb used in dredging a shallow stream. It consists of dynamite embedded in grout within a short pipe.

screwed into a collar at the upper end of the pipe. To the plug was attached a nipple or handle extending about 1 ft., and through a hole along the axis of the handle, the blasting wires were threaded, thus completing the assembly.

Details of the bomb are shown on the accompanying illustration.

In planting the bombs (from a rowboat) each unit was placed in a chuck or holders—a 2-in. pipe 10 to 12 ft. long—through which the blasting wires were drawn and left free. Next, the bomb was secured in the holder by means of a wire between the bomb handle and the top of the chuck. These preliminaries completed, the outfit was pushed down into the bed of the stream. The holding wire was cut, and the chuck was used to drive the bomb to the required depth.

The chuck was then carefully withdrawn, leaving the bomb planted in the bed of the river. As each bomb was planted, the free end of the blasting wires was attached to a float until ten or more bombs were set. Then the wires were connected in series, taped securely and joined to lead wires extending to a blasting machine set a safe distance away on the shore. All the charges were shot together.

The bombs were spaced 8 to 12 ft. apart, usually in the center of the channel, and side shots were used only where jutting rock ledges made them necessary. The channel shots loosened the stone and silt so thoroughly that much of the material was quickly carried away by the swift current. Large blocks of broken rock were maneuvered to the shore by hand, and the remaining burden was removed by scrapers.

The use of bombs eliminated the drilling of holes at a substanitial saving in labor and time. A crew of three men, operating in shifts on the boat, did all the necessary planting, while a fourth man handled the blasting machine. A crew with barge and scraper completed the job.

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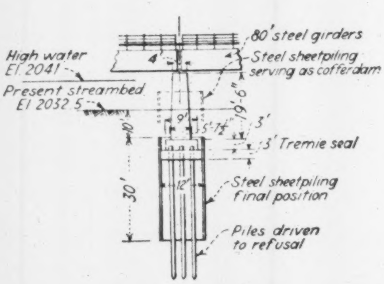
Sheetpiling Does Double Duty in Bridge-Pier Construction

THE HIGHWAY BRIDGE recently built across the Hassayampa River on U. S. 60 at Wickenburg, Ariz., consists of six 80-ft. steel deck girder spans on concrete piers and abutments. The west abutment and one pier rest on rock below streambed, but the other four piers and the south abutment are carried on timber foundation piles in the sandy riverbed. Ordinarily the concrete pier footings would have to be carried 25 ft. below streambed to protect the foundation piles from scour. In this case an alternative was used to save the expense of such deep foundations.

The bottom of the tremie seal was set arbitrarily 16 ft. below streambed. A sheetpile cofferdam (30-in. steel piles) was driven around the 12x26-ft. pier base. Then followed the excavation, the driving of the 40-ft. foundation piles, the pouring of the seal and the concrete pier itself. Finally, the steel sheetpiling was driven on down 10 ft. below streambed to protect the foundation piles against scour. The sequence of these operations is indicated in the accompanying sketch.



ELEVATIONS through bridge pier, showing the two positions in which steel piling served first as cofferdam and then to prevent scour around pile footings.

The easterly roadway approach fill spills around the east abutment and is protected from scour by a steel sheetpiling bulkhead roughly semi-circular in plan. The piles in this bulkhead had previously been used in the cofferdam for this abutment and for the pier, which was founded on rock. Thus all steel piles in the bridge served a temporary construction purpose and as integral parts of the completed structure.

This structure is a federal-aid project; the features of foundation construction noted in the foregoing were developed in the San Francisco office of the U. S. Bureau of Public Roads, of which Dr. L. I. Hewes is deputy chief engineer. George D. Whittle is bridge engineer.

Road Slide Repaired Quickly by Metal Cribwall

A LANDSLIDE on Barber Road near Boise, Idaho, blocked the highway and threatened to disrupt operations of a branch-line railroad. At this location the highway skirts around the toe of a steep hill on a 20-deg. curve. The railroad, a short distance downhill from the highway, is separated from it by a masonry wall.

A survey of the situation gave the state highway department two possible solutions: one, to build a retaining wall at the toe of the slope, at the inner edge of the road; the other, to move the road out beyond the sliding area. A combination of these two methods was finally adopted; a heavy rubble wall was built at the toe of the slide, using the large stone recovered from the slide material, and the road was shifted closer to the railroad track. To hold the relocated road, a metal crib retaining wall was built above the existing masonry wall adjacent to the track.

The overall height of the cribs ranged from 6 ft. 6 in. minimum at the two ends, to 15 ft. 10 in. maximum near the center. Forty-two cribs, each 8 ft. long, gave the wall an overall length of 336 ft.

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Letters to the Editor

Traffic on the Ohio River

Sir—In an article by Prof. John S. Worley on p. 52 of your issue of July 9, 1936, the following statement is made:

> At the time the improvement of the Ohio was proposed, which improvement has now been very largely completed, no study of the justification of such improvement was made and the operation on this river up to this time gives little encouragement that any justification will be found.

The writer, as U. S. District Engineer at Cincinnati, has been charged with the study of Ohio River traffic. It is believed that both parts of Professor Worley's statement are in error. The economics of the proposed Ohio River improvement were discussed at great length in the Report of the Special Board of 1906, printed in House Document No. 492, 60th Congress, 1st Session. The total tonnage in sight in 1906 was 9,350,000 tons. With improvement, the Board predicted an increase in tonnage sufficient to justify the public undertaking of the burdens involved.

The prediction has been verified. The official tonnage for the calendar year 1935 was 20,976,708. The commercial cost of moving non-metallic minerals by water is now less than one-third the rate for moving such products by rail; in the case of steel, the cost is less than one fourth the comparable rail rate. When the annual public costs are divided by the total commercial ton-mileage, a government cost per ton-mile is determined, which, added to the commercial cost, gives the total water cost per ton-mile. This total is uniformly less than the published rail tariffs. The writer not only believes that Ohio River justification will be found; he regards it as found already.

C. L. Hall
Lieut. Col., Corps. of Engineers.
Cincinnati, Ohio.
July 15, 1936.

Computing Concrete Strength

Sir—Mr. Mensch (*ENR*, April 30, 1936, p. 641) errs in his interpretation of my letter. I said that the flexural strength of concrete—computed by the straight line formula—was not the compressive strength given by the standard 6x12 cylinder.

The close confirmation of Mr. Mensch's parabolic distribution formula which he finds in the McCullough tests when the concrete strength is assumed to be 4,000 lb. per sq.in. is of course lacking in those specimens where the strength was not 4,000 lb. per sq.in. It would be lacking also if the compressive strength of this concrete had been measured in European fashion with 6-in. cubes. With these cubes the unit compressive strength is not the same as with our standard 6x12 cylinders. Which gives the "right" value? Which gives directly the flexural strength? Neither one of them does, nor is there aught but serious error in assuming that they do.

Since the straight line formulas are widely accepted and are established and since the 6x12 cylinder is the adopted test specimen, the only way to bring about a more reasonable relationship between allowed loads and ultimate loads is to increase working stresses.

Homer M. Hadley,
Regional Structural Engineer,
Portland Cement Association.
Los Angeles, California,
May 8 1936.

Wooden Transit 18 Years Old

Sir—In the issue of August 8, 1918, p. 283, you published a letter from W. J. Peters describing a homemade wooden transit built by Fred J. Meyer of Chehalis, Wash. The letter said, in part:

> The plumb bob is made from a 38-caliber revolver shell with a hole drilled for the string and soldered to an old spray nozzle. The bearing plate for the foot screws is a solid piece of 1½ in. fir, bolted through to prevent warping. The bearings for the foot screws are made of wood, lined with parts of a 12-gage shotgun shell. The instrument has a ball and socket shifting head.
>
> The one horizontal motion is attached to an old balance wheel from a sewing machine. This wheel serves to prevent warping of the plate, which is a solid piece of fir, 1½ in. thick, of octagonal shape, 7 in. across. The horizontal circle is made from an 8 in. celluloid circle, graduated to ½ deg.
>
> The center bearing is made up of a ½ in. gas pipe filled with babbitt metal and then drilled out to receive the solid steel spindle. The standards are of one piece fir, 1 in. thick, and substantially braced with four brackets.
>
> The telescope is 14½ in. long made up of small lenses from an old telescope, all contained in a 2 in. square piece of hollowed oak. A tangent screw is attached for levelling work, which has been done with an error of only ¼ in. in a quarter mile circuit.

The total cost of maintenance was $10.

Mr. W. J. Peters, who submitted the above letter in 1918, was reminiscing with the manager of a well known instrument company and mentioned the wooden transit made by Fred J. Meyer. The manager of the instrument company expressed a desire to borrow the transit to put in his show window in contrast with a present day instrument, but both agreed that it was hardly likely that the transit was still in a presentable condition.

However, an inquiry of Mr. Meyer revealed that the transit is still in daily use by a road contractor and is in perfect condition. The action is remarkably smooth, and no part of the wood was warped.

James O'Hearne
Portland, Ore.

Damaged River Gaging Station

Sir—The unusual character of the break in the concrete of the gaging station tower shown on page 50 of your issue of July 29 would indicate that there were no reinforcing rods in the concrete, as stated in the description accompanying the photograph.

I visited the site shortly after the flood and looking down about 35 ft. from the old bridge there were no reinforcing rods visible, only small stones. I visited the site again yesterday and went to the river bank where they are preparing to rebuild the tower. There are some rods showing about as indicated in the picture you reproduced, but they are ⅜ in. deformed bars and they stick up about 1 in. They look as if they had been cut with a shear and stuck into the concrete to hold it at the joint. I cannot see how the turning over of the tower would have produced the clean cut shown right at the joint.

The repairers cut out one side of the tower about 4 ft. up from the joint and no rods appear there going up vertically through the tower. What are shown to be reinforcing rods in your picture look like a few pins stuck into the concrete when it was poured.

Thos. Sheahan
Hagerstown, Md.
July 27, 1936.

Largest Liner Pier

Sir—In the June 11, 1936 issue, page 861, under the title "Second Pier in Super-Liner Terminal Goes Into Service," an account is given of the pier at which the "Queen Mary" of the Cunard-White Star Line docked upon the occasion of her maiden voyage, and a statement is made regarding the pier that "Its dimensions, 125 ft. wide and 1100 ft. long are unequalled in any port."

May I draw your attention to the fact that Erie Railroad Co.'s Pier No. 9 at Jersey City, N. J., across the river and downstream about three miles, exceeds these dimensions. This pier is 150 ft. wide and 1234 ft. long and was completed in 1928. It is of modern construction throughout, with a two-story steel frame shed 144 ft. wide, 1122 ft. long, 50 ft. high. It is used in part by the Dollar Steamship Co.

As the article in question deals with the scant attention paid to the new pier, because of the overwhelming interest in the new liner, it is appropriate to call attention to the Erie pier, since it has also been forgotten—even by an engineering journal.

H. J. Stroebel
Cleveland, Ohio
July 2, 1936

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August 20, 1936

Asphalt Creed Upset

SUCCESSFUL MENDING of the Galveston jetties with hot asphaltic concrete deposited in water, and the planned use of that material in greater mass and depth of penetration for the Columbia River jetties, is a notable development in the use of asphalt. As described in this issue, few accomplishments in asphalt construction have come with greater surprise. In roofing and paving, the use of hot oils and asphalt mixtures in the presence of water has for years called for a forbidding clause in engineering specifications. No incompatibility of engineering materials and water has been more nearly an accepted creed. Even when a few years ago the engineers on the lower Mississippi proved the possibility of molding asphalt sheets and placing them hot for underwater bank revetment the experience was not considered conclusive; the molding of the hot material was still a dry operation although immersion followed before cooling had set up the bitumen. At Galveston the full step has been taken of depositing the hot material under water in bulk to settle, take form and bond up under its own weight aided by vibration. This the mixture used has done with pronounced success and a new construction method has been added to those available for underwater work.

Go to the People

MAINE AND CALIFORNIA, a continent apart, propose this fall to put the question of road-tax diversion to the vote of the people. In common with scores of other states they have learned that legislatures and state executives are not dependable guardians of road-tax money for, when pressed by other expenses, they lack the courage to finance by independent levies. With this year included, over half a billion dollars diverted on the plea of depression will have been the exaction from taxes paid in the last half decade for highway improvement but not a dollar of which has gone into construction or maintenance. No twist in interpretation of the tax law nor pretense of borrowing the money can conceal the fact that road users in five years have been mulcted for half a thousand millions under false pretenses. The only way effectually to stop this is to ask action by the people and on their decision write honest laws to enforce it. Past practices of petitioning and legislative lobbying have been mostly expensive failures; the diversion interests are masters of the roadbuilders in these practices. California is going to the people for a constitutional amendment and Maine is asking a referendum vote on a specific statute. The ways of action are different but the purposes are the same—appeal to the fundamental sources of law for direct action. This is the only way to get permanent action on tax diversion.

Get Men Off WPA

PAY INCREASES are being given to WPA mechanics on Brooklyn Navy Yard projects to keep them from going into private jobs, thus reducing the efficiency of the WPA work. If the statements credited to A. J. Brehm, director of the Navy Yard operations, are correct, the Navy has a quite different attitude toward WPA from that expressed by the President. The President only recently reiterated his earlier statement that the whole object of the Works Progress Administration was to give men useful work until such time that private industry could reabsorb them. In this respect WPA work is in quite a different class from normal construction or even PWA projects; the *work in hand* is not the main objective, the objective is to provide *employment* on the work. When the need for employment ends the job should end. Mr. Brehm says the Navy looks on the deferred maintenance work that it has been doing with WPA money as a works program, not a relief program. That is the chief danger in WPA; it is too easy to get into the habit of looking on it as a normal way of getting public work done at little cost to the local agency. Establishment of that habit must be opposed. PWA operations must be considered as emergency work to be abandoned as rapidly as regular work can be found for the men so employed. Rather than increase the pay of men to hold them on PWA they might better be paid a premium for landing a job in private industry.

Superior Through Its Simplicity

REVISED PLANS for the East River vehicle tunnel at New York were made public on Aug. 15. They are worthy of special note for the simple arrangement of the Manhattan approach and exit plazas. Proposals to drive vehicle tunnels under the East River in the neighborhood of 38th Street have been under consideration for several years, but heretofore no plan has presented a satisfactory solution of the problem created by the nearness of the deep channel that must be passed under to the relatively high ground of the avenues that it is necessary to reach on the east side of Manhattan. Further, all plans have had to take into consideration the posibility of a future crosstown tunnel. The early plans for the East River tunnel either reached the surface by involved spiral loops or gained the surface by ramps that removed the exit and entrance portals far to the north or south of 38th St., the point where connection must be made with the future crosstown tunnels. Even the initial plan of the present tunnel authority resorted to the use of looped ramps. In the revised plans they are abandoned. The river is crossed at an angle, the tubes passing under the navigable channel just south of 42nd St., instead of opposite 38th St., and swing south under First Ave. to gain the needed distance before coming to the surface. The engineers of the tunnel authority are to be congratulated for having evolved so satisfactory a solution of their problem. Its very simplicity makes it hard for the uninformed to realize how difficult was the task that contfronted them when they took over the project.

Prospects Not Bright

A PLAN TO ELECTRIFY 12,000 miles of railway lines at a cost of $600,000,000 has been put forward by the Federal Power Commission in one of a series of studies of future power requirements that it is making. The work would be spread over twenty railroads, mostly in the East, on sections of double track that had a traffic density of 1,000,000 gross tons a month in the period 1926-29. Electrification of a much greater mileage of rail lines than now is electrified frequently was put forward before the depression began, not as a means for developing a greater power market but in the belief that economies would come therefrom in addition to the improvement in service that electrification would bring. But practical railroad men were not receptive to the proposals. The great initial investment was a deterring factor, and experience with existing electrified lines had shown that electrification does not pay except where traffic is very dense. Then too, the displaced steam locomotives, many with a long useful life, had to be written off or sold at a loss, adding to the capital cost. At the present time, improvements have increased the efficiency of steam locomotives, thus reducing the margin in favor of electrification, and some think that more radical improvements are still to come. There also is the diesel engine with its possibilities still largely unknown. Hence the prospects are not bright for the plan of the power commission.

Zoning Pays

TWO LARGE CITIES in the Middle West, Milwaukee and Detroit, are debating the pros and cons of city zoning. Milwaukee, after sixteen years' experience with a fairly good zoning ordinance, is making an intensive study of existing land use with a view to modernizing the law so as to bring it into consonance with the most recently developed city plan. Improving the functioning of the city as a whole is aimed at, rather than aggrandizement of favored property owners. Detroit, on the other hand, never has had any zoning ordinance. Such protection against undesirable development as has been obtained has come from deed restrictions in the better residential districts. For many years the plan commission periodically has made strenuous efforts toward getting a zoning ordinance adopted.

Examination of the development of these two cities indicates that both methods of land use restriction have been effective, yet both have errors to atone for. Neither has been successful in neutralizing the forces of big-city blight, which were working long before zoning came along. Big-city zoning is likely to be low grade; effective zoning is almost impossible to maintain because city government systems usually do not foster good zoning. In addition to knowledge of city growth processes; good zoning requires neighborhood interest and responsiveness to neighborhood desires, and this latter is rare in big cities.

Such lack of responsiveness is not offset by deed restrictions, for these have one major shortcoming, namely that change is almost impossible unless 100 per cent agreement is obtained. This may be as detrimental to logical growth as entire lack of control. The small town, because of its unity, is likely to have good zoning. The likelihood decreases with increasing size, until in cities like Chicago the other extreme is reached and many times more area is zoned for industrial and commercial uses than can ever be so occupied.

Nevertheless big-city zoning is worth while regardless of its handicaps and the frailties of the zoners, for it does in many cases give legal protection against harmful land use, a protection that has been upheld by high court decisions. Milwaukee therefore is wise in revamping its 16-year-old laws, and Detroit may well start now to curb the greed of those who would misplace commercial enterprises in the midst of residential districts, none too well protected by deed restrictions. Despite its many drawbacks, zoning pays.

Safe Building

DISCUSSION OF THE ISSUE of safe building following the recent collapse of an apartment house under construction in New York City brings to attention once more the importance of competence and responsibility. It is conceded by all that safe building—and for that matter sanitary and firesafe building—is a matter of public interest. It is recognized also that buildings cannot be safe where ignorance prevails; competence in design and construction is indispensable. These principles are generally accepted, but views are diverse as to how they should be interpreted.

Some hold that the owner, because his money is at stake, is the controlling interest to deal with, and that he may and should be trusted to employ those who have the necessary skill. Others hold that the owner cannot be trusted to do so, and, therefore, that he should be required by law to employ architects, engineers and contractors whom the public authorities know to be qualified to plan and direct building. Still others hold that the only assurance of safe building lies in municipal inspection, and that this inspection should be so close and thorough as to take care of even incompetent or dishonest work.

It is important that such divergent opinions be reconciled. Existing practice follows none of the three views cited, but is a compromise composite of all three: It allows anyone to direct building, however lacking in knowledge or integrity; on the other hand it virtually requires a substantial degree of competence in design (but not in construction), through the municipal examination and approval of plans; and finally it provides a rather cursory inspection of the actual construction. The practice is known to be imperfect, and in consequence the assurance of safe building remains a problem today just as it was in the days of Hammurabi.

The history of building laws is largely a record of the struggle against the greed and recklessness of those who build; every basic provision of building codes, fire laws and tenement house regulations represents a victory over the desire of owners to build cheaply, ignorantly and badly. The choice, then, lies between obtaining competence and responsibility by limiting the direction of building to duly qualified persons, or, instead, imposing a system of municipal supervision and inspection that will assure good work even in the absence of competent direction.

To those familiar with building procedure and inspection there can be no question as to the choice. Municipal building inspection is subject to limitations that make it wholly inadequate to assure good building, and ways to extend it to provide rigid, detailed inspection of all work, even were that feasible, would defeat their own objective by penalizing the conscientious and careful builder to the benefit of the shoddy and irresponsible builder. Moreover, efforts to make the city building department capable of giving such a complete supervision would result in that department absorbing a large part of the independent architects, engineers and builders.

There is no natural and practicable way to safe building other than to place it in the hands of those equipped with knowledge, skill and experience to carry it on properly. The entrusting of design and supervision to recognized architects and engineers is one necessary step. The other is to restrict construction or the direction of construction, to qualified builders. When such men, and no others, direct building and certify to its adequacy we may expect to have the nearest approach to a safe building system that it is possible to obtain.

CURRENT NEWS

A. F. of L. Predicts Much House Building

Shortage of 13,196,000 homes predicted in business survey—Others think estimate too high

A HIGH LEVEL of home building by private industry is foreseen by the American Federation of Labor. In its August survey of business the A. F. of L. expresses the opinion that the gains made in building construction during 1935 and thus far in 1936 "has strengthened confidence throughout the business world." Unnamed authorities are quoted as estimating that 3,250,000 homes are needed to replace houses no longer fit for habitation. Further replacement from 1936 to 1945 accounts for another 3,250,000, while 6,750,000 homes would be needed between now and 1945 for those who will start homes in that period. The total is estimated at 13,196,000 new homes.

Ten-year program

To keep up with the need, says the A. F. of L., "We should build 1,320,000 new homes each year from now until 1945." In the A. F. of L. survey it is stated that the largest number of homes ever built by private industry in a single year was 840,000 in 1928 and that the average for 1920 to 1929 was 646,000 yearly. In 1935 only 300,000 new homes were built, says the survey. Then the question is asked "Can the industry supply the nation's need?" In an attempt to answer the question the A. F. of L. estimates that private industry can provide only 880,000 of the 1,320,000 new homes needed each year, since homes for incomes under $1,500 a year in the north, and $1,200 in the south, cannot be built at a profit. To meet that situation the A. F. of L. expresses the opinion that the Wagner housing act, which failed of passage at the last session of Congress, must be enacted into law.

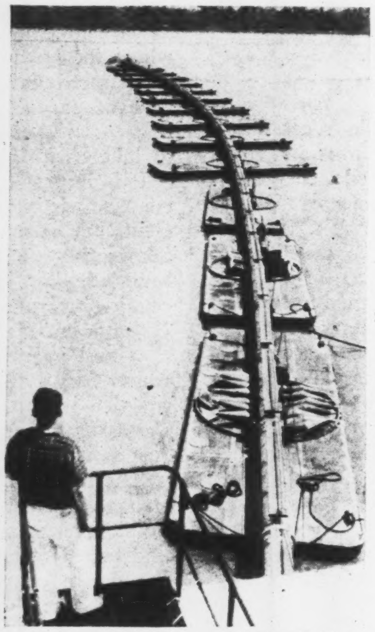
Issuance of the A. F. of L. survey has prompted comment in other quarters. The Federal Housing Administration estimates that building in a normal year in the past has provided housing for 470,000 families. According to FHA estimates, home building in 1932 provided for only 27,000 families; in 1934 for 22,063; in 1935 for 55,810. If the rate maintained thus far in 1936 continues throughout the year FHA estimates that housing for 111,000 families will have been constructed.

Shortage of skilled labor

At the Associated General Contractors headquarters it was pointed out that the limiting factor in home-building at present is a shortage of skilled labor. The A.G.C. cites an instance where a brick layer 81 years old has been pressed into service. The Chamber of Commerce of the United States estimates that 400,000 units per annum will be required for the next ten years. In other Washington quarters, doubt was expressed that home building could be brought to a maximum rate without systematic efforts to overcome sales resistance built up by knowledge of the difficulties which overtook many home owners during the depression.

Ten Eyck Contract Awarded

Contract for the construction of the superstructure of the Ten Eyck housing project in Brooklyn, N. Y., has been awarded by PWA to Starrett Bros. & Eken of New York City on their low bid of $7,556,400. This is the largest housing project in the country. It will provide 1,614 apartments.



Wide World Photo

KEEPING THE RIVER OPEN

Discharge line of a government dredge working 24 hours a day to keep the Mississippi River open to traffic. With river stages lower than ever before in August, more than 3 ft. below zero at St. Louis, eight government dredges are busy removing sand bars.

Illuminating Engineers To Discuss Highway Lighting

A number of papers on highway lighting are included in the program of the 30th convention of the Illuminating Engineering Society, which is to be held at Buffalo, N. Y., from Aug. 31 to Sept. 3.

The papers on highway lighting will be: The use of refraction and reflection in street luminaires, by Thomas W. Rolph, chief engineer of the Holophane Co.; Visibility studies on lighted highways, by Kirk M. Reid and H. J. Channon, Nela Park engineering department of the General Electric Co.; Discussion of the code of highway lighting, by Preston F. Millar, president of the Electrical Testing Laboratories, of New York City; Lighting the highways for safety, by Earl J. Reeder, traffic engineer of the National Safety Council.

All these papers will be read Sept. 3.

PWA Policy

Washington Correspondence

AT A RECENT press conference, the President amplified his views on the new restrictions on PWA grants. He said that the federal government would be willing to make a grant of 45 per cent if the local agency is in a position to use that amount for relief labor. This means, he said, that a project needing a large amount of unskilled labor would be able to get the entire 45 per cent grant. If the project is of a character requiring a large amount of skilled labor, it probably would mean that the applicant could qualify for only a small part of the 45 per cent.

The President explained that it would be necessary to widen the field from which labor is drawn because of the scarcity of skilled labor on relief rolls. He said there will be no geographical limit on the recruiting of skilled labor.

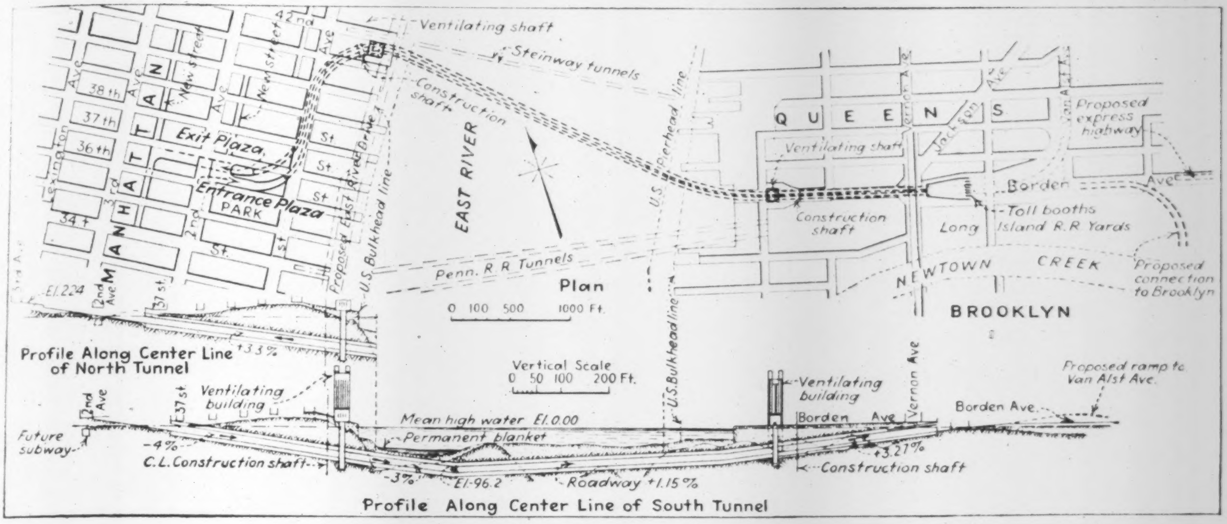
Grant based on relief labor

The President stated that there is no objection to the municipality using as much of its own money as it wishes for skilled labor, but that the federal grant will be limited to the amount of relief labor that can be absorbed. He said there were a few projects that might be excepted from the ruling because of the fact that they had been virtually approved before the new order went into effect. He repeated that the government will reimburse the municipality for every dollar that they can spend, up to 45 per cent, in taking people off relief rolls. He also pointed out that the municipality may borrow, and that the money obtained from the loan can be used to hire skilled labor. Asked as to the conditions under which a municipality might borrow from PWA, the President said that loans would be extended if the municipality could show that reasonable interest rates were not obtainable on loans from other sources or that adequate local bank credit was not available.

Transfers authorized from WPA

The President then was asked if a PWA project would be allowed to take skilled labor away from WPA projects even if it were essential to the WPA. The President said that if skilled labor was on the WPA rolls, it could be requisitioned for a PWA project. He explained that WPA is attempting to get more accurate information concerning the specialized skills of those on its rolls. He also pointed out that any worker transferred from a WPA project to a PWA project would then be paid the prevailing wage of the locality.

Ickes said on Aug. 18 that no start can be made on the new three hundred million PWA program until the President clarifies rules and policies. He has had no direct instructions and is still uncertain as to the interpretation of the new policy. If the Federal grant is limited to direct labor cost only, it will amount to an average grant of about 10 per cent with a maximum of 33 per cent. Previous PWA experience shows that only


42nd
Ventilating shaft
Steinway tunnels
Construction shaft
Ave
New Street
New Street
Ave
Ave
Lexington
38th
37th
36th
34 t
3rd Ave
MANHATTAN
Exit Plaza
Entrance Plaza
PARK
St
St
St
St
St
2nd
Proposed East River Drive
U.S. Bulkhead line
EAST RIVER
U.S. Pierhead line
Penn. R.R. Tunnels
Plan
0 100 500 1000 Ft.
U.S. Bulkhead line
QUEENS
Vernon Ave
Jackson Ave
Van Alst Ave
Proposed express highway
Ventilating shaft
Borden Ave
Construction shaft
Toll booths
Long Island R.R. Yards
NEWTOWN CREEK
Proposed connection to Brooklyn
BROOKLYN
El. 224
2nd Ave
37 st
+3.3%
Profile Along Center Line of North Tunnel
Vertical Scale
0 50 100 200 Ft.
2nd Ave
37 st
Ventilating building
Mean high water El. 0.00
Permanent blanket
Future subway
-4%
C.L. Construction shaft
-3%
El -96.2
Roadway +1.15%
Ventilating building
Borden Ave
Vernon Ave
Borden Ave.
Proposed ramp to Van Alst Ave.
+3.27%
Construction shaft
Profile Along Center Line of South Tunnel

PROJECTED VEHICLE TUNNEL UNDER THE EAST RIVER AT NEW YORK

33 per cent of total project cost goes for direct labor on site.

Recent project approvals totalling $22,000,000 are not under the new program as is generally supposed, but will be financed from remains of old ERA funds, leaving no additional balance unallocated. Those projects now in the office of the comptroller general for approval will get under way as soon as passed upon. No applications have been received for projects with grants of less than 45 per cent. Ickes admitted that the new program has been delayed by lack of definite instructions from President, but he hopes to get these next week.

◆

War Department Recommends Deepening N. Y. Harbor

The Board of Engineers for Rivers and Harbors has transmitted to the Senate Committee on Commerce a recommendation that the current plans for improvement of N. Y. Harbor be modified to provide a channel 2,000 ft. wide, suitably widened at the bends, from the Atlantic Ocean to West 59th Street in Manhattan, with depths of 45 ft. in the outer harbor and in the Hudson River Channel as far north as W. 40th St., and a depth of 48 ft. as far as W. 59th St. The cost of the work is estimated at about $2,481,000, with an extra annual maintenance cost estimated at $50,000.

The report stated that, although the present channel depths and widths are adequate for most of the harbor traffic, the *Queen Mary* and the *Normandie* experience considerable difficulty. These ships may draw as much as 40.5 ft. When the keels of the ships are close to the river bottom, and the ships are crosswise of the river, they dam the tidal flow and create currents which tend to get them out of control.

The recommended channel depth would not endanger the existing tunnels under the river, since they do not exceed the present depth over many sections of the tunnel and since ample cover would remain over them.

Work to Start Soon on Vehicle Tunnel

Plans adopted for tunnel under East River at New York—First construction bids to be called in September

THE NEW YORK CITY Tunnel Authority, on Aug. 15, announced that it had approved revised plans, prepared by its engineering department, for the proposed 38th Street tunnel under the East River at New York, and that the first bids on the work would be called as soon as the revised plans had been approved by the War Department and Public Works Administration. It is hoped that ground for the project can be broken about Sept. 15.

As originally proposed, the two tubes of the vehicle tunnel left Manhattan Island in the vicinity of 38th Street, passed under the East River, and came to the surface in the borough of Queens, Long Island, near the intersection of Jackson and Borden Avenues. Because the land in Manhattan rises fairly steeply from the river, the design of approaches that would give convenient access to the streets on the east side of Manhattan presented an unusually difficult problem. A further factor to be considered was the proposed 38th St. cross-town tunnel in Manhattan to connect the Midtown tunnel, now under construction under the Hudson River, with the proposed East River tunnel. Earlier plans kept the tunnel portals near 38th St. by the use of spiral ramps.

New location selected

In the revised plans, to gain distance in coming to the surface in Manhattan, the alignment of the river tunnels has been swung north so that the tunnels cross the river at an angle, passing under the pierhead line just south of 42d St. Entering Manhattan island, the two tunnels swing south under First Ave. and then swing west to come to the surface between First and Third Aves., south of 37th St.

The entrance plaza will be at street grade at Second Ave. The exit plaza will pass under Second Ave. coming to the surface between Second and Third Avenues in a new street to be cut through from 34th St. to 41st St. This street will be depressed to pass under 36th St., an eastbound street leading toward the entrance plaza. For approaches to the entrance plaza, Second Ave. is to be widened from 34th to 36th St., and a new street is to be cut through from the plaza to 40th St.

A very simple scheme for connection with the proposed 38th St. crosstown tunnel is understood to have been worked out, but the details are not ready to be made public.

Proposed deepening of the navigable channel in the East River has called for a deeper crossing than heretofore planned.

The present plan calls for bringing the two tubes to the surface in Queens east of Vernon Ave. on a line with Borden Ave. Beyond that point the matter of connections to the main arterial highways in Queens and in Brooklyn is subject to negotiation between the New York City Tunnel Authority and the presidents of the two boroughs.

Tunnel design

The design of the tunnel is similar to that of the Holland and Midtown Tunnels, two 31-ft. diameter cast-iron tubes with a 21-ft. roadway widened to 24 ft. on curves and a clear headroom of 13 ft. 6 in. The shallower sections of the approaches will be cut and cover of typical subway construction. Ventilation will be by the same system that has worked so successfully in the Holland tunnel, there being one ventilating shaft on each side of the river. Construction of the underwater section will be by the use of shields working from construction shafts just back of the ventilation shaft on each side of the river. This will permit an early start on the ventilating buildings and will keep the ventilating equipment from being the controlling factor in the opening of the tunnel as was the case on the Holland tunnel. The total length of the tunnels, portal to portal, will be 6,369 ft.

Driving of the tunnels under the East

River is expected to be difficult because of the character of the bottom which is made up chiefly of rock ledges and porous sand that will not hold air as well as the soft mud of the Hudson River.

Specifications for the construction shaft in Queens are ready, so that bids can be called as soon as the revised plans have been approved. It is hoped that this work can start by Sept. 15. The call for bids on the main tunnel must be delayed until additional borings have been obtained along the new line of the tunnel. Borings opposite 38th St. were taken by the Board of Transportation several years ago but fuller information is needed to prepare the contracts, drawings, and specifications for the revised alignment.

The work is to be financed by the Public Works Administration through a grant of $11,235,000 and a loan of $47,130,000 secured by bonds of the tunnel authority to be retired from tolls.

The New York City Tunnel Authority consists of Alfred B. Jones, chairman, William H. Friedman and Albert T. Johnston. Ole Singstad, former chief engineer on the Holland Tunnel, is chief engineer, William McK. Griffin, former division engineer of the Board of Transportation, is deputy chief engineer, James H. Dugan is engineer of design and Jacob Mechanic is engineer of construction. Jesse B. Snow, chief engineer of the Board of Transportation, the J. G. White Engineering Corp. and Gibbs & Hill, Inc., have acted as consultants on the tunnel; John C. Evans, R. B. Stoeckel and C. J. Tilden have acted as special consultants on the approaches.

◆

WPA Allots Four Million For Repair of Flood Damages

The WPA has allotted $4,228,377 for reconstruction of flood damaged bridges in Pennsylvania, Maine, Massachusetts and West Virginia.

The allotments include $2,292,997 for building or repairing 79 bridges in Pennsylvania; $1,053,400 for 22 bridges in Maine; $769,480 for six bridges in Massachusetts; and $172,500 for four bridges in West Virginia.

An allotment of $30,000 has been made for the construction of concrete public buildings in Miami which may be used as hurricane shelters.

◆

Road Builders Promote Safety on the Highways

In view of the increasing number of motor vehicle accidents, the American Road Builders' Association has decided to appoint a Committee on Safe Highways to promote the building of safety into highways. The association is convinced that the straightening and widening of roads, separation of lanes of traffic, flattening of curves, reduction of grades, and relocation of some routes will eliminate many conditions that are a major contributing factor in highway accidents. Chas. Upham, engineer-director of the association, states that the committee now being organized will cooperate with other agencies interested in highway safety.

FIFTH DREDGE FOR FORT PECK DAM

Conceived in the inner sanctum of the dredging superintendent's office, born amid the deafening bedlam of the Fort Peck shops, and ignominiously launched by being dunked by a crane, the dredge *Galloping Coulee* proudly joined the now-famous Fort Peck fleet one fine July day, coming to rest in the middle of the dike core pool. The new and fifth unit of the fleet differed from its mates only in design, size and application. Its sole purpose in an extremely short life was to remove a few sand streaks that were carried into the silt core of the dike when a violent storm broke the dike levees and let loose the core pool (*ENR*, June 25, 1936, p. 933). The new dredge is really a 6-in. sandsucker, powered by a tractor motor and mounted on a small scow. As is the case with all new equipment, the first day's operations were nothing but grief, but after the crew became accustomed to its idiosyncracies, the dredge did yeoman service, successfully removing every bit of the objectionable sand. Here is the first day's official dredging report: Material pumped—*Gallatin*, 38,560 cu.yd.; *Jefferson*, 39,789 cu.yd.; *Madison*, 43,44 cu.yd.; *Missouri*, 45,863 cu.yd.; *Galloping Coulee*, one bushel.

Washington Highlights

By Paul Wooton

The District of Columbia court of appeals has issued a temporary restraining order halting work on the Jefferson Memorial Park at St. Louis until a decision is reached on a case, now pending in the court, which seeks to stop the project permanently on the ground that the relief act, under which federal money was provided for the memorial, is unconstitutional (*ENR*, July 2, 1936, p. 29).

The Florida East Coast Railway has petitioned the Interstate Commerce Commission for authority to abandon its 125-mile Key West Branch. The railway states that repair of the damage done by last September's hurricane would cost $1,800,000, an amount which is not justified by the traffic carried.

At the request of the Tennessee Valley Authority, the Interstate Commerce Commission has dismissed a petition filed by TVA asking approval for the abandonment of the 11.2-mile Lafollette branch of the Southern Ry., partly located within the area flooded by the Norris Dam.

The branch line is parallelled by tracks of the Louisville & Nashville outside the flooded area. An agreement has been reached between the two railroads and TVA providing that Southern trains will use L. & N. tracks in maintaining service, and giving TVA a complete release from all liabilities in return for a payment of $125,000. Reconstruction of the flooded trackage would have cost an estimated $350,000, an expenditure not justified by the amount of traffic involved.

Taking aerial photographs of various watersheds will be the first step by the Department of Agriculture in the $5,000,000 survey of watersheds which the last Congress authorized it to undertake. The survey is for the purpose of gathering more data as to the exact areas that could profitably be planted in trees or grass to aid in erosion control. Attention is to be given to the possibility of water storage in the low lying areas.

Generation of power at Boulder Dam will be begun officially on Sept. 7 when President Roosevelt, at the conclusion of his address to the World Power Conference, will press a button which will open the needle valves at the dam and start one of the generators.

The electrical impulse starting operation will be transmitted from Washington to the dam by telegraph wires, and the effect of turning on the generator will be relayed back to the Power Conference by radio.

◆

New York City WPA Raises Pay For Architects and Engineers

Architects, engineers, and technical men to the number of 7,000 have been granted pay increases by Col. Brehon B. Somervell, new WPA Administrator at New York. The increases bring the scale of wages for these men up to the prevailing-rate basis as prescribed by the Relief Act of 1936. The new rates are based in part on the results of a study made in 1934 by the Metropolitan Regional Committee on Engineers' Employment and Salaries and in part on existing civil service rates. It is about 25 per cent below that now paid technical men in civil service. The new rates range from $1,440 per year for rodmen to $2,600 for supervising draftsmen and assistant supervising engineers.

The increases come as a result of long continued efforts to have technical men employed by the WPA placed on the same basis of pay accorded to skilled mechanics.

SOCIETY CALENDAR

CANADIAN GOOD ROADS ASSOCIATION, annual meeting, Charlottetown, P.E.I., September 1-3.

THIRD WORLD POWER CONFERENCE, Washington, D. C., Sept. 7-12.

PUBLIC WORKS CONGRESS, joint meeting, American Society of Municipal Engineers and International Association of Public Works Officials, Toronto, Ont., September 28-30.

CANADIAN INSTITUTE OF SEWAGE AND SANITATION, Royal York Hotel, Toronto, Oct. 1-2, 1936.

INTERNATIONAL CONFERENCE ON STEEL CONSTRUCTION, Berlin, Germany, Oct. 2-7.

NATIONAL SAFETY CONGRESS, annual meeting, Atlantic City, N. J., Oct. 5-9.

INTERNATIONAL ASSOCIATION FOR BRIDGE AND STRUCTURAL ENGINEERING, second congress, Berlin, Germany, Oct. 11.

NATIONAL SOCIETY OF PROFESSIONAL ENGINEERS, annual convention, Washington, D. C., Oct. 12-13.

NATIONAL COUNCIL OF STATE BOARDS OF ENGINEERING EXAMINERS, annual meeting, Knoxville, Tenn., Oct. 19-21.

AMERICAN PUBLIC HEALTH ASSOCIATION, annual meeting, New Orleans, La., Oct. 20-23.

ENGINEERING INSTITUTE OF CANADA, maritime general professional meeting, St. Johns, N. B., Aug. 27-29.

AMERICAN WATER WORKS ASSOCIATION, Rocky Mountain Section, Denver, Colo., Sept. 21-23.

NEW ENGLAND WATER WORKS ASSOCIATION, annual convention, New York, N. Y., Sept. 22-25.

AMERICAN WATER WORKS ASSOCIATION, Wisconsin Section, Waukesha, Wis., Oct. 6-7.

AMERICAN WATER WORKS ASSOCIATION, Southwest Section, section meeting, Fort Smith, Ark., Oct. 12-13.

AMERICAN WATER WORKS ASSOCIATION, New York Section, fall meeting, Saratoga Springs, N. Y., Oct. 15-16, 1936.

AMERICAN WATER WORKS ASSOCIATION, New Jersey Section; NEW JERSEY WATER WORKS ASSOCIATION; and the SOUTH JERSEY ASSOCIATION OF WATER SUPERINTENDENTS; joint meeting, Atlantic City, N. J., Oct. 23-24.

AMERICAN WATER WORKS ASSOCIATION, Missouri Valley Section, Kansas City, Mo., Oct. 26-28.

AMERICAN WATER WORKS ASSOCIATION, North Carolina Section, Charlottle, N. C., Oct. 26-28.

AMERICAN WATER WORKS ASSOCIATION, Virginia Section, Charlottesville, Va., Oct. 29-31.

EXAMINATIONS for registration as Engineers, Land Surveyors and Architects will be held as follows:

NEW MEXICO, examination for professional engineers and land surveyors, Santa Fe, Sept. 12.

◆

Drought Relief Projects

The PWA has made two allotments for irrigation and power development in the drought area of Nebraska.

One allots a loan of $916,000 and a grant of $749,000 to the North Loup River Public Power and Irrigation District of Ord, Neb. The project will make the waters of the North Loup River available for the irrigation of about 40,000 acres in the district between Taylor and the city of North Loup. It is expected that the electric power provided by the project will enable some of the farms lying above the canal to irrigate by the use of pumping plants. Of the total loan and grant, $1,387,000 will be used for the irrigation work, while $278,000 will be used for the construction of power facilities.

The other allotment is a loan of $765,000 and a grant of $625,909 to the Middle Loup Public Power and Irrigation District of Custer and Sherman Counties, Neb. This project will use the water of the Middle Loup River, which has an uncommonly uniform flow, for the irrigation of 30,000 to 40,000 acres of arid land. The projects cover territory extending along the Middle Loup River from Sargent to Loup City. Power will also be generated on this project. About $1,040,000 of the allotment will be used for the irrigation work, and about $350,000 will be used for power facilities.

The transmission lines of the two systems will be connected to provide a loop transmission system which will furnish power from two directions, insuring reliable service.

◆

Brief News

Bids for a New Set of the terminal grain elevators being constructed by the Argentine Government (*ENR*, April 23, p. 614) will be called for shortly. The cost of this section of the program will be about $5,000,000. Bids will be called within two months for another group of elevators to cost about $2,500,000.

The City Planning Commission of Cincinnati, Ohio, has approved the application of the Ring Construction Co., Washington, D. C., for permission to erect a group of 94 apartment buildings, containing 840 apartments, in Cincinnati. The group, known as the Colonial Village, will cost $3,000,000. Harvey Warwick, of Washington, is the architect.

Flood Control Projects in Oklahoma, approved by the U. S. Engineers, are being held up by the lack of state funds for land purchases required. At a recent popular referendum a proposed $500,000 appropriation for the State Conservation Commission, intended to be used for the purchase of land, was voted down. $6,097,900 worth of projects are involved.

President Cardenas of Mexico has ordered the Ministry of Communications and Public Works to start surveys and prepare estimates for the construction of two highways that will connect central Mexico with the state of Sonora. One road will run from Mexico City, the other from Guanajuato City. The roads will join at Indé in Durango State; a single road will run from there to Hermosillo, the capital of Sonora.

The War Department has approved an application by the Rockland-Westchester Hudson River Crossing Authority for approval of revised plans of a fixed bridge to be constructed across the Hudson River between Tarrytown, N. Y., and Grandview-on-Hudson, N. Y. The revised plan provides for a navigation opening having a horizontal clearance of 1,000 ft. and a minimum vertical clearance of 155 ft. above mean high water. The proposed bridge would be over 3 miles long. Construction of the bridge was authorized by an act of Congress approved in August, 1935.

Personals

Wallace H. Challburg has been appointed city engineer of Wahoo, Neb.

George E. Bean, assistant city engineer of Milwaukee, has been appointed city manager of Escanaba, Mich.

Major Roy B. Burdick has been appointed chief of operations at the Memphis district of the U. S. Engineers.

W. Fred Starks, county engineer of Nassau County, N. Y., since 1922, has resigned to go into private practice.

A. G. Stanley, a district engineer for the Idaho Highway Department, has been appointed state construction and maintenance engineer.

Albert Gastrock, for the past three years office manager of the Pennsylvania office of PWA has been appointed assistant state director of the PWA.

Karel Tonglet, deputy city engineer of Monrovia, Calif., since 1923, has been made chief engineer of the municipal water department.

H. L. Walton and W. M. Mackenzie have been elected to the board of directors of Smith, Hinchman & Grylls, Inc., architects and engineers, of Detroit.

Carl A. Hoglund has been appointed principal engineer of the inventory and pricing branch of the bureau of valuation, Interstate Commerce Commission at Washington, D. C.

Col. John J. Kingman, U. S. District Engineer at Boston since 1934, has been appointed division engineer for the Pacific Southwest with headquarters at San Francisco.

Robert Kingery has resigned his position as director of the state department of public works, of Illinois, to return to a position as director of the Chicago Regional Planning Commission.

Grover P. Dillman has been appointed president of the Michigan College of Mining and Technology. At his inauguration he received the degree of doctor of engineering. Dr. Dillman was highway commissioner of Michigan from 1929 to 1933.

Ross C. Boas, who has been regional director of the WPA for the states of Pennsylvania, New Jersey, Delaware, Maryland, Virginia, and West Virginia, has been appointed chief engineer of the PWA in Pennsylvania.

Hez K. Johnson, who was recently chief designer for the National Engineering & Distribution Co. of Chillicothe, Mo., has been appointed resident engineer inspector for PWA at Willow Springs and West Plains, Mo.

Col. Philip B. Fleming, engineer in charge of the Passamaquoddy project, has been transferred to the Rural Resettlement Administration at Washington, D. C. Capt. Samuel D. Sturgis, Jr., will direct the concluding activities at Passamaquoddy.

Thomas H. Cutler has resigned his position as chief engineer of the Missouri State Highway Department to accept a position as chief engineer of the Kentucky State Highway Department.

Mr. Cutler's name was incorrectly given as C. H. Cutler in the Aug. 13 issue.

Obituary

EUGENE J. MCMANUS, for 20 years building inspector for the city of Erie, Pa., died there on Aug. 12.

W. P. FUNSTON, a construction engineer of Little Rock, Ark., died in that city, aged 64, on Aug. 7.

TYLER FIELD, president and founder of the Ferro Concrete Construction Co. of Cincinnati, Ohio, died in Cincinnati on Aug. 13 at the age of 61.

SAMUEL N. GOODMAN, 36, government engineer on the Muskingum Conservancy Project was killed on Aug. 4, when struck by a crank, while inspecting a gate hoist at the Charles Mill Dam.

JAMES CANTWELL, a building contractor in Elizabeth, N. J., died there on Aug. 12 at the age of 88.

OSCAR G. ALLEN, 53, a contractor of Washington, D. C., died in Seaside Heights, N. J., on Aug. 12.

ALFRED A. TITSWORTH, formerly dean of engineering at Rutgers and recently professor of mathematics there, died at Penn Yan, N. Y., on Aug. 16. Professor Titsworth had been on the faculty of Rutgers since 1886.

WILLIAM J. COMSTOCK, 67, president of the Buffalo Construction News, of Buffalo, N. Y., died in that city on Aug. 8.

A. H. DUNNINGTON, 78, a retired building contractor of Muncie, Ind., died there on Aug. 5.

WILLIS M. BUNGY, a contractor of Cleveland, Ohio, died in Cleveland Heights on Aug. 6.

CONSTRUCTION STATISTICS FOR THE WEEK

CONTINUED high federal awards, offset by a lull in private work, combine to raise this week's engineering construction awards above last week and above the average week this year. The total for this week is $50,072,000 made up of private, $5,521,000; public, $44,551,000, of which federal is $16,720,000 and state and municipal, $27,831,000. Corresponding values for a year ago are: total, $29,438,000; private, $2,956,000; public, $26,482,000; federal, $15,184,000; state and municipal, $11,298,000.

Highway awards at $16,470,000 set the highest week's volume on record in this classification since October 19, 1933. Losses for this week, however, are reported in sewerage, bridges, industrial buildings, commercial buildings and earthwork, drainage and waterways, with large gains shown by waterworks, public buildings, unclassified and highways. The amounts in each classification are: industrial buildings, $3,274,000; commercial buildings, $522,000; public buildings, $13,433,000; highways, $16,470,000; bridges, $1,674,000; sewerage, $1,581,000; waterworks, $2,391,000; earthwork, drainage, $7,575,000; unclassified, $3,152,000.

The large projects for the week include: factory buildings, Ciba Pharmaceutical Products, Inc., at Summit, N. J., $500,000; four-stand tandem mill for production of tin plate, Republic Steel Corp., Niles, Ohio, $1,000,000; medical clinic building, Indiana University, Indianapolis, Ind., $504,000; junior high school, East Orange, N. J., $704,000; jail at Reservation Point, San Pedro, Calif., $879,000; superstructure, Ten Eyck Housing Project, Brooklyn, N. Y., $7,556,000; highways: Michigan, $3,947,000; Mississippi, $1,032,000; Georgia, $2,019,000; Missouri, $659,000; New York, $706,000; North Dakota, $504,000; Texas, $882,000; transformers for main pumping plants, Metropolitan Water District, Los Angeles, Calif., $717,000; sewerage system, Divisions O, N, P, Buffalo Sewer Authority, Buffalo, N. Y., $1,350,000; two 20-inch pipe line dredges, U. S. Engineers, St. Paul, Minn., $1,201,000; constructing turbo-electric seagoing dredge "Goethals," U. S. Engineers, New York, N. Y., $3,385,000; dam No. 12, Mississippi River, Bellevue, Iowa, $2,224,000; 113 mi. steel pipe line, Peoples Natural Gas Co., Pennsylvania, $1,500,000 est.

New capital for the week consists of state and municipal bond sales, $8,261,000; new corporate security sales, $69,000; and PWA loans and grants, $3,056,000.

CONTRACTS

(Thousands of Dollars)

	Weekly August 1935	Average Prev. 4 Weeks	Week Aug. 20 1936
Federal Government	$12,611	$13,008	$16,720
State and Municipal	12,396	19,273	27,831
Total public	$25,007	$32,281	$44,551
Total private	6,439	12,751	5,521
Week's total	$31,446	$45,032	$50,072
Cumulative to date: 1935.....$875,260		1936....$1,492,577	

NEW PRODUCTIVE CAPITAL

(Thousands of Dollars)

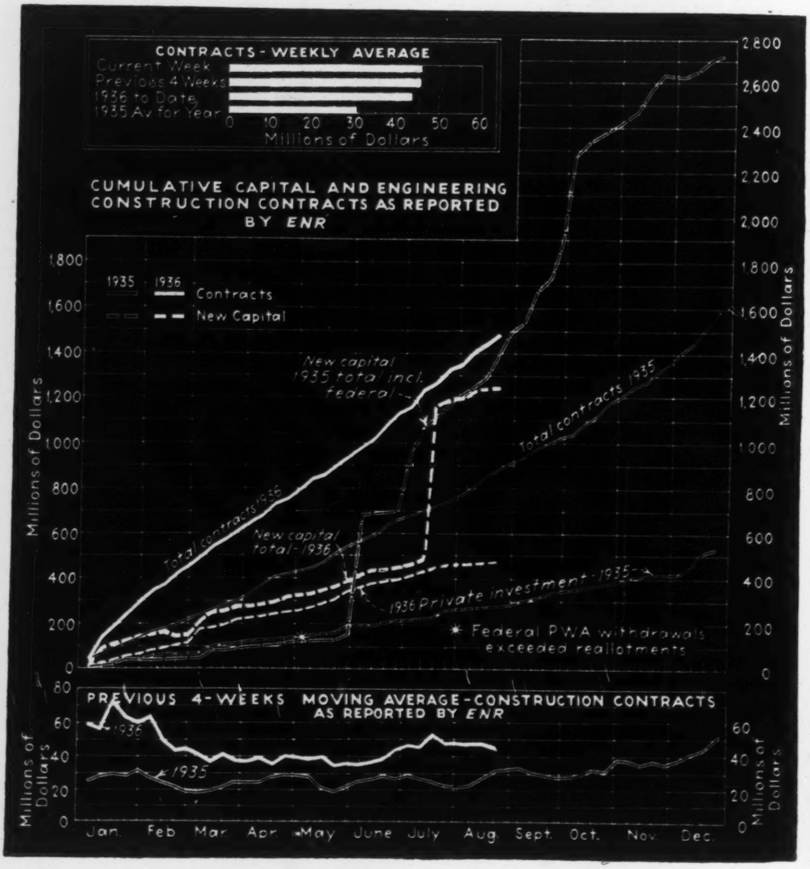
1936	Week Aug. 20	Cumulative
State and municipal	$8,261	$326,456
PWA non-federal	3,056	59,289
RFC loans	....	12,479
Corporate issues	69	151,716
PWA loans, Private	....	*—1,665
Total Non-Federal	$11,386	$548,275
Federal	....	691,237
Total new capital	$11,386	$1,239,512
Cumulative to date: 1935....$1,407,022	1936....$1,239,512	

*Bond sales in this classification exceed reallotments during current year.

Note: These figures include private bonds, and stocks sold for productive purposes; state and municipal bonds for construction; PWA loans and grants to states and municipalities. RFC direct purchases of bonds for "self liquidating" projects, and 25 per cent of WPA appropriations for construction projects.

INDEX NUMBER

ENR Cost	1913 = 100	1926 = 100	ENR Volume	1913 = 100	1926 = 100
Aug., 1936	**208.10**	**100.03**	**July, 1936**	**188**	**82**
July, 1936	204.40	98.25	June, 1936	194	85
Aug., 1935	195.10	93.82	July, 1935	96	42
1935 (Av.)	195.22	93.84	1935 (Av.)	135	58
1934 (Av.)	198.10	95.23	1934 (Av.)	114	50
1933 (Av.)	170.18	81.80	1933 (Av.)	102	47


CONTRACTS - WEEKLY AVERAGE
Current Week
Previous 4 Weeks
1936 to Date
1935 Av. for Year
0 10 20 30 40 50 60
Millions of Dollars
CUMULATIVE CAPITAL AND ENGINEERING CONSTRUCTION CONTRACTS AS REPORTED BY ENR
1935 1936
Contracts
New Capital
New capital 1935 total incl. federal
Total contracts 1935
Total contracts 1936
New capital total - 1936
1936 Private investment - 1935
* Federal PWA withdrawals exceeded reallotments
Millions of Dollars
0 200 400 600 800 1,000 1,200 1,400 1,600 1,800 2,000 2,200 2,400 2,600 2,800
PREVIOUS 4-WEEKS MOVING AVERAGE - CONSTRUCTION CONTRACTS AS REPORTED BY ENR
1936
1935
0 20 40 60 80
Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec.